PROCEEDINGS

2ND INTERNATIONAL CONFERENCE ON CIVIL ENGINEERING FOR SUSTAINABLE DEVELOPMENT

14 ~ 16 February, 2014

ICCESD

ICCESD 2014

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Proceedings

ICCESD-2014

2nd International Conference on
Civil Engineering for Sustainable Development

14-16 February, 2014
Khulna, Bangladesh

Proceedings

ICCESD-2014

2nd International Conference on Civil Engineering for Sustainable Development

14-16 February, 2014, Khulna, Bangladesh

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ICCESD-2014

2nd International Conference on Civil Engineering for Sustainable Development

Chairman of the Conference

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Secretary of the Conference

Dr. Muhammad Harunur Rashid
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Preface

We are very pleased to introduce the proceedings of the 2nd International Conference on Civil Engineering for Sustainable Development (ICCESD-2014) which took place in the Department of Civil Engineering, Khulna University of Engineering & Technology, Khulna-9203, Bangladesh, during February 14-16, 2014. The overall goal of this conference was to promote applied research and professional activities relevant to different branches of Civil Engineering for the sustainable development of Bangladesh to face the great challenges of the 21st century.

The Civil Engineering discipline is the oldest and vast discipline that includes the broader spectrums of Environmental, Geotechnical, Structural, Transportation and Water Resources Engineering. ICCESD always aims at bringing the Researchers, Scientists, Engineers, and Scholars together in all the areas of Civil Engineering, and provides an ideal platform for sharing and exchanging professional, technical and academic knowledge on all aspects of Civil Engineering. We hope and strongly believe that the conference will open a window of exchanging the scientific and technical information among all the participants from home and abroad. At the same time, it is expected to create a common platform for sharing the ideas among the Engineers and Academicians from the developed and developing countries in the world having different experiences in research skills, practices and standards. We also believe that it will promote the international collaboration on sustainable technology in all the branches of Civil Engineering with respect to global warming and climate change. Considering the enormous scope of Civil Engineering discipline, a wide range of thematic areas and expertise are included in the conference.

In the event, the conference was highly successful. More than 268 pre-registered authors submitted their works in the conference. ICCESD-14 finally accepted and hosted 171 original research papers, after a double blinded peer review process. During the conference 23 technical sessions were held in order to advance and contribute to specific research areas in the field of Civil Engineering.

Finally, it is appropriate that we record our thanks to our fellow members of the Technical and Organizing Committee for their work in securing a substantial input of papers from both in and abroad and in encouraging participation from those areas. We are also indebted to those who served as chairmen in the technical sessions. Without their support, the conference could not have been the success that it was. We also acknowledge the authors themselves, without whose expert input there would have been no conference. We would also like to express our appreciation for the contribution of the Civil Engineering Division, IEB specifically for the logistical arrangements in the conference. Their efforts made a great contribution to its success. We also acknowledge the important contributions of the Editorial Committee in editing and assembling the conference proceedings. The sponsoring facilities offered by Bandar Steel Industries Ltd., Seven Circle Cement Ltd. and Bashundhara Cement are highly appreciated and acknowledged for the successful completion of the conference.

The continuing success of this conference series means that planning can now proceed with confidence for the next event to be held in the same place in 12-13 February, 2016.

Prof. Dr. Md. Abul Bahsar
Conference Chair, ICCESD-2014
Department of Civil Engineering
Khulna University of Engineering & Technology
Khulna-9203, Bangladesh

Conference Programme Outline for ICCESD-2014

Collection of Conference Kit:

February 13, 2014 (from 15:00~18:00)

Venue: Civil Engineering Building, KUET

Day 1: 14 February, 2014 (Friday)

Time	Venue	Events
8:30-9:15	CE Building	Collection of Conference Kit and Reporting of Registered Participants
9:30-10:25	KUET Auditorium	Inaugural Ceremony of ICCESD-2014
10:25-10:45	KUET Auditorium	Refreshment
		Keynote Session:
10:45-11:45	KUET Auditorium	Keynote Speech by Prof. Dr. Shamim Z. Bosunia
		Keynote Speech by Prof. Dr. M. Feroz Ahmed
11:45-14:30	CE Building	Friday prayer & Lunch
14:30-16:06	CE Building	Parallel Technical Sessions (1st, 2nd & 3rd)
16:06-16:35	CE Building	Coffee Break
16:35-17:59	CE Building	Parallel Technical Sessions (4th, 5th & 6th)
17:59-18:30	CE Building	Prayer Break
18:30-19:30	CE Building	Parallel Technical Sessions (7th, 8th & 9th)
19:30-	CE Building	Dinner at CE Building

Day 2: 15 February, 2014 (Saturday)

Time	Venue	Events
8:30-10:06	CE Building	Parallel Technical Sessions (10th, 11th & 12th)
10:06-10:40	CE Building	Coffee Break
10:40-12:28	CE Building	Parallel Technical Sessions (13th, 14th & 15th)
12:28-14:10	CE Building	Lunch & Prayer Break
14:10-14:30	CE Building (Room A)	Keynote Speech C
14:30-16:06	CE Building	Parallel Technical Sessions (16th, 17th & 18th)
16:06-16:40	CE Building	Prayer & Coffee Break
16:40-18:04	CE Building	Parallel Technical Sessions (19th, 20th & 21st)
18:04-18:18		Prayer Break
18:18-19:42	CE Building	Parallel Technical Sessions (22nd & 23rd)
19:42		Depart to Conference Dinner
20:15-22:15	Hotel City Inn	Closing Ceremony of ICCESD-2014 & Conference Dinner

CONFERENCE PROGRAMME

1st Technical Session Session Chair: Prof. Dr. M. Feroz Ahmed, VC Stamford University, Bangladesh		Environmental Engineering-I		Venue: Seminar Room A Date: 14/02/2014
Time	**Paper ID**	**Paper Title**	**Page No.**	**Authors Name**
14:30~14:42	E 1435	Available studies on groundwater for drinking purpose in Khulna city: A review	pp. 001-002	Q. H. Bari, M. Wahiduzzaman, K. M. Hassan and S. M. Moniruzzaman
14:42~14:54	E 1416	Evaluating the existing condition of clinical waste management in Khulna city and its possible health hazards	pp. 003-004	Abu Sayed Mohammad Akid, Kh. Mahbub Hassan, Omar Shahrear Apu, Sm. Arifur Rahman and Md. Azizur Rahman
14:54~15:06	E 1442	Study on composting of uncooked kitchen waste	pp. 005-006	S. M. Moniruzzaman, Joyita Adhikary, Q. H. Bari and Kanij Fahmida
15:06~15:18	E 1439	Design of user friendly eco-sanitary latrine	pp. 007-008	Istiaque Mahmud and Md. Niamul Bari
15:18~15:30	E 1445	Plastic waste recycling scenario in Khulna city	pp. 009-010	A.S.M. Riyad, Abu Zakir Morshed, Kh. Mahbub Hassan and Sk. Farid Hossain
15:30~15:42	E 1436	Environmental comparison between stone and brick chips construction	pp. 011-012	Md. Shahrior Alam, Syed Ishtiaq Ahmad
15:42~15:54	E 1440	Study on the locally available clayey soils for the preparation of liner	pp. 013-014	Shuma R. Saha and Muhammed Alamgir
15:54~16:06	E 1464	Development of a hybrid water treatment system using solar still cum sand filter with ceramic media	pp. 015-016	Abdullah Al Sadeek and Khondoker Mahbub Hassan
16:06~16:35	**Prayer & Coffee Break**			

2nd Technical Session Session Chair: Prof. Dr. Monsur Rahman, BUET		Water Resources Engineering-I		Venue: Seminar Room B Date: 14/02/2014
Time	**Paper ID**	**Paper Title**	**Page No.**	**Authors Name**
14:30~14:42	W 1415	Storm water management for urban areas of Bangladesh by analytical & modelling approach: A case study of Chalna municipality	pp. 091-092	M. Rahman, Rupayan Saha, M. M. Haque and Shahadat Hossain
14:42~14:54	W 1441	Application of CCHE2D mathematical model in the Gorai offtake for two-dimensional simulation	pp. 093-094	Nishan Kumar Biswas and Muneer Ahammad
14:54~15:06	W 1454	Computation of discharge and flow volume for different flooding scenario in the lower Meghna estuary	pp. 095-096	Ali Mohammad Rezaie, Sumaiya, Shah Alam , Ishtiaq Ahmed, Hafez Ahmed, Md. Nurul Kadir, Muhammad Khalid Bin Siddique, Mansur Ali Jisan and Anisul Haque, Munsur Rahman
15:06~15:18	W 1447	Hydro-morphological change near old Brahmaputra Off take	pp. 097-098	Fahmida Noor, Md. Musfequzzaman and Md. Abdul Matin
15:18~15:30	W 1448	Macroscopic two-phase flow modeling of concentrated suspensions of waste water using a continuum constitutive equation	pp. 099-100	Shimul Hazra
15:30~15:42	W 1464	Hydraulic and morphological impact assessment of capital dredging in the Ganges river using Hec-Ras	pp. 101-102	Imran Khan
15:42~15:54	W 1405	Two-dimensional simulation of flows in bends of an open channel by iRIC Nays2D	pp. 103-104	Md. Hasibur Rahman Lemon, Md. Shahjahan Ali and Md. Abdul Qaiyum Talukder
15:54~16:06	W 1428	Geostatistical analysis of groundwater level fluctuations in the shallow aquifer of northwestern Bangladesh	pp. 105-106	S.K. Adhikary, A.A. Sharif, S.K. Das and G.C. Saha
16:06~16:35	**Prayer & Coffee Break**			

3rd Technical Session Session Chair: Prof. Dr. Md. Monjur Hossain AUST		Structural Engineering-I		Venue: Seminar Room C Date: 14/02/2014
Time	**Paper ID**	**Paper Title**	**Page No.**	**Authors Name**
14:30~14:42	S 1427	Determination of setting time from the electrical resistivity of cement paste	pp. 177-178	Md. Rokon Hasan, Muhammad Harunur Rashid, S M Arifur Rahman
14:42~14:54	S 1416	Study on compressive strength of concrete using stone dust as partial replacement of fine aggregate	pp.179-180	M.S. Amin, A.H.M.G. Hyder, M.R. Hassan and S.M.Z. Islam
14:54~15:06	S 1417	Strengthening of existing RC roof slab	pp. 181-182	Md. Rakibul Hasan, Md. Abu Sufian Talukder and Muhammad Harunur Rashid
15:06~15:18	S 1421	Comparative study on rapid chloride durability tests on supplementary concretes	pp. 183-184	S. J. U. Ahmed
15:18~15:30	S 1428	Behavior of Ferro-cement and RC column under lateral and axial loading	pp. 185-186	Md. Abu Sufian Talukder, Md. Rakibul Hasan and Muhammad Harunur Rashid
15:30~15:42	S 1436	Laboratory investigation of no fines concrete	pp. 187-188	Md. Iftekar Alam, Mir Abdul Kuddus and Shariful Islam
15:42~15:54	S 1437	Necessity of non-linear bond stress-slip model for simulating the cyclic behavior of Reinforced Concrete (RC) members	pp. 189-190	Sharmin Reza Chowdhury and Wahid Hassan
15:54~16:06	S 1438	Strength deterioration of brick aggregate concrete in NaCl environment	pp. 191-192	Md. Saiful Islam, Sabrina Islam, Md. Moinul Islam and Md. S. Z. Basunia
16:06~16:35	**Prayer & Coffee Break**			

4th Technical Session		Environmental Engineering-II		Venue: Seminar Room A
Session Chair: Prof. Dr. Quazi Hamidul Bari, AUST				Date: 14/02/2014
Time	**Paper ID**	**Paper Title**	**Page No.**	**Authors Name**
16:35~16:47	E 1472	Urbanization and sustainability challenges in Dhaka city, Bangladesh	pp. 017-018	Muhammad Abdullah, Sushil Kumar Das and Oyama Tatsuo
16:47~16:59	E 1473	Trend analysis of climatic extreme events for the Haor area of Bangladesh using climatic indices	pp.019-020	Saniruzzaman
16:59~17:11	E 1424	Environmental impact assessment on Kalishuri to Surjomoni union road construction project at Bauphal, Patuakhali	pp. 021-022	Md. Hamidul Islam, Md. Shafiqul Islam and Tania Yeasmin
17:11~17:23	E 1432	Study on Environmental Management Plan (EMP) of proposed leather industry in Lebukhali, Patuakhali	pp. 023-024	Md. Hamidul Islam, Chhanda Rani and Rownak Jahan
17:23~17:35	E 1441	Generation of ammonia in deliming operation from tannery and its environmental effect: Bangladesh perspective	pp. 025-026	Md. Abul Hashem, Ahidul Islam, Subrata Paul and Shamima Nasrin
17:35~17:47	E 1426	Graphene based reverse osmosis desalination technique: Cost and Energy perspective	pp. 027-028	Debashis Sarker, Khondoker Mahbub Hassan, Moythely Roy and Nazia Zerin
17:47~17:59	E 1469	Characteristics and management of commercial solid waste in Khulna city of Bangladesh	pp. 029-030	A S M. Riyad, Kh. Mahbub Hassan, Md. Azizur Rahman, Mahabub Alam and Abu Sayed Mohammad Akid
17:59~18:30	**Prayer Break**			

5th Technical Session		Water Resources Engineering-II		Venue: Seminar Room B
Session Chair: Dr. Engr. Gholam Mostofa, Chairman, CE Division, IEB				Date: 14/02/2014
Time	**Paper ID**	**Paper Title**	**Page No.**	**Authors Name**
16:35~16:47	W 1436	A Study on Water Logging Reduction in Chittagong City	pp. 107-108	Shoukat Ahmed, Syed Abdullah Mohit and Aysha Akter
16:47~16:59	E 1456	Engineering activities and their mismanagement at halda: A unique river for natural spawning of major Indian carps	pp. 109-110	Md. Humayain Kabir, Mohammed Monzoorul Kibria, Iftakharul Alam Russell and Mohammad Mosharraf Hossain
16:59~17:11	W 1432	Hazard Assessment for Flash Flood in Deeply Flooded Haor Basin of Bangladesh	pp. 111-112	Sabbir Hossain, Anisul Haque, B. Bhattacharya, M.F.A Khan and M. Maswood
17:11~17:23	W 1444	Erosion Trend Analysis at Chandpur Confluence of Meghna Estuary	pp. 113-114	Md Hafez Ahmed , Ali Mohammad Rezaie , Muhammad Khalid Bin Siddique and Ishtiaq Ahmed
17:23~17:35	W 1434	Application of GIS in risk assessment of storm surge in Bhola district	pp. 115-116	Mutasim Billah, Umme Kulsum Navera and Farhana Noor
17:35~17:47	W 1412	Analytical assessment of groundwater recharge potential for the Ganges-Kobadak (G-K) irrigation project area	pp. 117-118	S. M. Shahed Sharif and Sajal Kumar Adhikary
17:47~17:59	W 1425	Cost of irrigation water in Bangladesh: A study from Chittagong and Rangpur district	pp. 119-120	Md. Reaz Akter Mullick, Manjura Musharraf Anannya and H.M. Emrul Ahmed
17:59~18:30	**Prayer Break**			

6th Technical Session		Transportation Engineering-I		Venue: Seminar Room C
Session Chair: Prof. Dr. Md. Mazharul Haque, BUET				Date: 14/02/2014
Time	**Paper ID**	**Paper Title**	**Page No.**	**Authors Name**
16:35~16:47	T 1418	Determining suitable bus stop location using geographic information system in Kaptai road, Chittagong	pp. 249-250	Md. Rezuanul Islam, Tanveer Rahman , Sourav Das , Sohaib Sinan , Khatun-E-Zannat and Md. Rabiul Islam
16:47~16:59	T 1422	Estimation of the highway capacity in Khulna metropolitan city	pp. 251-252	M.T. Hasan and Q.S.Hossain
16:59~17:11	T 1423	A study on the factors involved in truck accidents in Bangladesh	pp. 253-254	Ishtiak Ahmed, Abu A. Sufian and Sakib M. Khan
17:11~17:23	T 1424	Evaluating the performance of a road: A case study of Muradpur to Dewanhat road, Chittagong	pp. 255-256	Md. Rezuanul Islam, Shoukat Ahmed, and Debasish Roy Raja
17:23~17:35	T 1421	iRAP star ratings for national highways in Bangladesh	pp. 257-258	M A Rahman and M M Hoque
17:35~17:47	T 1429	BRT as mass transit option: The context of Dhaka	pp. 259-260	Md. Mazharul Hoque, S. M. Abdullah Al Mamun and Hasib Mohammed Ahsan
17:47~17:59	T 1426	Pedestrians in sustainable urban transport management and climate mitigation: The case of megacity Dhaka	pp. 261-262	Hasinae Jannat, Tusar K. Roy and Kamrul H. Sohag
17:59~18:30	**Prayer Break**			

7th Technical Session		Environmental Engineering-III		Venue: Seminar Room A
Session Chair: Prof. Dr. Md. Saiful Islam, KUET				**Date: 14/02/2014**
Time	**Paper ID**	**Paper Title**	**Page No.**	**Authors Name**
18:30~18:42	E 1408	Feasibility of rooftop rainwater harvesting in coastal areas: A case study	pp. 031-032	A. S. M. Asif Iqbal , and Kh. Mahbub Hassan
18:42~18:54	E 1453	Reuse of textile dying wastewater for irrigation in cultivation of some selected crops: A review	pp. 033-034	Khandaker S, Saha G.C, Islam M. S and Ratan M.M.R
18:54~19:06	E 1457	Present scenario of rainwater harvesting in Rajshahi	pp. 035-036	Md. Niamul Bari, K. M. Nahid Hossain Shaon, Fatema Khatun, H.M. Rasel and Sobura Khatun
19:06~19:18	E 1454	Effects of salinity on land fertility in coastal areas of Bangladesh	pp. 037-038	Md. Azizur Rahman, Kh. Mahbub Hassan, Mahabub Alam, Abu Sayed Mohammad Akid, A S M Riyad
19:18~19:30	E 1412	Climate induced vulnerabilities in Natun Bazar Char areas: Water supply and sanitation perspective	pp. 039-040	Selim Ahmed, Kh. Mahbub Hassan , Q. H. Bari and Md. Hasibur Rahman Lemon
19:30~	**Dinner at CE Building**			

8th Technical Session		Water Resources Engineering-III		Venue: Seminar Room B
Session Chair: Prof. Dr. Kh. Md. Shafiul Islam, KUET				**Date: 14/02/2014**
Time	**Paper ID**	**Paper Title**	**Page No.**	**Authors Name**
18:30~18:42	W 1431	Establishment of co-relation between remote sensing based TRMM data and ground based precipitation data in North-East region of Bangladesh	pp. 121-122	Munshi Md. Shafwat Yazdan, Ahmmed Zulfiqar Rahaman, Farhana Noor and Bushra Monowar Duti
18:42~18:54	W 1421	A study on variation of temperature and rainfall in north-central region of Bangladesh	pp. 123-124	Rezanur Rahman
18:54~19:06	W 1410	Study of precipitation variability in Bangladesh: An index based approach	pp. 125-126	Feroj Alam Evan and Md. Shahjahan Ali
19:06~19:18	W 1463	An assessment of best fitted rainfall distribution for projected climate change over Bangladesh using statistical downscaling method	pp. 127-128	Supria Paul, Mohammad Alfi Hasan and Ahsan Azhar Shopan
19:18~19:30	W 1455	Freshwater corridor in South-West region of Bangladesh and impact of SLR	pp. 129-130	M.A. Matin, Md. Mashfiqul Islam, Ashfia Siddique and Md. Mafizul Islam
19:30~	**Dinner at CE Building**			

9th Technical Session		Structural Engineering-II		Venue: Seminar Room C
Session Chair: Prof. Dr. Md. Keramat Ali Molla, KUET				**Date: 14/02/2014**
Time	**Paper ID**	**Paper Title**	**Page No.**	**Authors Name**
18:30~18:42	S 1447	Performance of fly ash concrete in seawater against freeze-thaw action	pp. 193-194	Md. Moinul Islam and Md. Saiful Islam
18:42~18:54	S 1450	Effectiveness of color coating on corrosion of steel reinforcement in concrete	pp. 195-196	Bappa Kumar Paul, Muhammad Harunur Rashid and Milon Kanti Howlader
18:54~19:06	S 1411	Performance evaluation of RC buildings designed by BNBC 2013	pp. 197-198	Md. Abdullah Al Sarfin and Tahsin Reza Hossain
19:06~19:18	S 1413	Cost optimization of columns	pp. 199-200	T. Das and M.S. Bari
19:18~19:30	S 1423	Comparison of wind load among BNBC and other codes in different type of areas	pp. 201-202	R. M. Faysal
19:30~	**Dinner at CE Building**			

10th Technical Session		Environmental Engineering- IV		Venue: Seminar Room A
Session Chair : Dr. Md. Niamul Bari, RUET				Date: 15/02/2014
Time	**Paper ID**	**Paper Title**	**Page No.**	**Authors Name**
8:30~8:42	E 1421	Removal of CR(VI) from aqueous solution using bone char as a low cost adsorbent	pp. 041-042	A.H.M.G. Hyder , S. A. Begum and N. O. Egiebor
8:42~8:54	E 1425	Study on decentralized wastewater treatment plantin Paanchtala colony Khulna	pp. 043-044	Raisa Khan, Md. Tanvir Ishtiak Shaon and Muhammed Alamgir
8:54~9:06	E 1406	Development of an arsenic and iron removal unit using sand filter with charcoal and iron powder	pp. 045-046	Tamanna Tabassum Tanni, Kh. Mahbub Hassan and Kanij Fahmida
9:06~9:18	E 1447	Impact of water logging on agriculture and food security: A case study in Satkhira, Bangladesh	pp.047-048	Md. Ajmal Hossain Gazi and S. M. Moniruzzaman
9:18~9:30	E 1415	Characterization, flow dynamics and treatment of waste water of textile dyeing vat at Kushtia district in Bangladesh	pp. 049-050	S.M.Hasan Razu, Abul Kalam Azad, Q. H. Bari and Masudur Rahman
9:30~9:42	E 1459	Medical Waste Management at Hospitals in Dhaka City: A case study	pp. 051-052	Zerin Binte Alam, Md. Ashiqur Rahman, Muhammed Alamgir and Khondoker Mahbub Hassan
9:42~9:54	E 1431	Speciation of copper in water and potential of application of copper salt for algae removal at the saidabad water treatment plant	pp. 053-054	Shamsunnahar Suchana and M. Ashraf Ali
9:54~10:06	E 1433	Surface water salinity intrusion assessment of South-West region of Bangladesh: A GIS based approach	pp. 055-056	Ishtiaq Ahmed, Md. Nurul Kadir Seum and Md. Hafez Ahmed
10:06~10:40	Coffee Break			

11th Technical Session		Water Resources Engineering-IV		Venue: Seminar Room B
Session Chair: Prof. Dr. Md. Shajahan Ali, KUET				Date: 15/02/2014
Time	**Paper ID**	**Paper Title**		**Authors Name**
8:30~8:42	W 1423	Impact of mixed cropping on groundwater based irrigation in South-West region of Bangladesh	pp. 131-132	Shamim Mohammad Murshid
8:42~8:54	W 1472	Response of coastal structures due to wave loading	pp. 133-134	M. Ahammad Sharif and Iftekhar Anam
8:54~9:06	W 1417	Sustainable navigability improvement of Mongla Port	pp. 135-136	Motiur Rahman
9:06~9:18	W 1456	Effect of climate change on flood inundation of the Jamuna river basin	pp. 137-138	Md. Mostafizur Rahman and Mohammad Mostafa Ali
9:18~9:30	W 1426	The livelihood vulnerability indices to assess the climate change induced risks -a case study of coastal Bangladesh	pp. 139-140	Md. Tanjim Islam, md. Bellal Hossen and Md. Shahjahan Ali
9:30~9:42	W 1453	Predicting bankline changes in the karnafuli river using ARCGIS	pp. 141-142	Rana Das, Md. Billal Hossen and Aysha Akter
9:42~9:54	W 1443	Assessment on the adequacy of navigation of some inland waterway routes of Bangladesh	pp. 143-144	Supria Paul, M. Abdul Matin and Ahsan Azhar Shopan
9:54~10:06	W 1424	Predicting non revenue water through leakage detection in a distribution system	pp. 145-146	Md. Apel Mahmud and Aysha Akter
10:06~10:40	Coffee Break			

12th Technical Session		Transportation Engineering-II		Venue: Seminar Room C
Session Chair: Prof. Dr. Quazi Sazzad Hossain, KUET				Date: 15/02/2014
Time	**Paper ID**	**Paper Title**	**Page No.**	**Authors Name**
8:30~8:42	T 1420	Analysis of bus network coverage for Dhaka city along with its service quality	pp. 263-264	Hasib Mohammed Ahsan, S. M. Abdullah Al Mamun, Rakibul Hassan and Tanjinur Akter
8:42~8:54	T 1409	Increasing air traffic demand and relevant issues in Bangladesh	pp. 265-266	Hasib Mohammed Ahsan, Md. Eleous, Md. Emrul Hasan, Md. Sharikur Rahman and Fahim Ahmed
8:54~9:06	T 1410	Estimation of road traffic accidents cost in Khulna Metropolitan City	pp. 267-268	N.J. Imi, Q.S. Hossain and N. Islam
9:06~9:18	T 1414	Transportation scenario of Dhaka City: Congestion and Air Pollution	pp. 269-270	Sadia Binte Salam ,Priyanka Das and Tanweer Hasan
9:18~9:30	T 1415	Road safety assessment for pedestrians on Dhaka-Aricha Highway (N5)	pp. 271-272	Ahmed Md. Hafez Hoque and Md. Mazharul
9:30~9:42	T 1411	Capacity evaluation of roundabout intersections in Khulna Metropolitan City using SIDRA	pp. 273-274	N. Islam, Q. S. Hossain and N. J. Imi
9:42~9:54	T 1416	Rail safety at level crossings in Bangladesh	pp. 275-276	Hasib Mohammed Ahsan, Mutamid Billah Azzacy, and Md. Shajedul Islam
9:54~10:06	T 1417	Significance of derailments in rail safety in Bangladesh	pp. 277-278	Hasib Mohammed Ahsan, Md. Shajedul Islam and Mutamid Billah Azzacy
10:06~10:40	Coffee Break			

13th Technical Session		Environmental Engineering-V		Venue: Seminar Room A
Session Chair: Prof. Dr. Muhammed Alamgir, VC, KUET				Date: 15/02/2014
Time	**Paper ID**	**Paper Title**	**Page No.**	**Authors Name**
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11:28~11:40	E 1428	Impact assessment of cyclonic storm surges on ecoystem services in the South-West coastal zone of Bangladesh	pp. 065-066	Ahsan Azhar Shopan, G.M. Tarekul Islam, A.K.M. Saiful Islam, Md. Munsur Rahman, Attila N. Lazar and Craig Hutton
11:40~11:52	E 1430	Study on the performance of different coagulants for turbidity and color removal	pp. 067-068	S. M. Moniruzzaman, Mehedi Hasan Mishuk, Quazi Shammas Sayeed and Md. A. T. Fahmidur Rahman
11:52~12:04	E 1422	Evaluation of CR(VI) removal efficiency from wastewater using Bio-Char	pp. 069-070	A.H.M.G. Hyder , S. A. Begum and N. O. Egiebor
12:04~12:16	E 1446	Assessment of grey water use potential, a case study	pp. 071-072	Juthika Roy and Sudip Kumar Pal
12:16~12:28	E 1401	A preliminary survey of residents for the project to prevail tss in parbayarjhapa, Bangladesh	pp. 073-074	Tomohiro Umemura, Kh. Md. Shafuil Islam, Md. Saiful Islam, Mika Hasegawa, Yukinori Kusaka, Hiroaki Terasaki and Teruyuki Fukuhara
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10:52~11:04	G 1422	Statistical evaluation of bearing capacity of Khulna sub-soil	pp. 297-298	RajibBanik, NazmaKhatun, Masum Shaikh and Md. Keramat Ali Molla
11:04~11:16	G 1430	Investigation on strength development of cement stabilized organic soil	pp. 299-300	Arojit Kumer Das, Md. Zahidul Islam, Gopal Chandra Saha, Chinmoy Dutta and M. M. Tariq Morshed
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11:52~12:04	G 1427	Effect on permeability and shear strength with the variation of grain size of sand	pp.307-308	Sohana Afreen and Md. Abul Bashar
12:04~12:16	G 1428	“Geo textile” –a tremendous invention of geo technical engineering	pp. 309-310	Md. Eanamul Haque Nizam and Subrata Chandra Das
12:16~12:28	G 1425	Performance of admixture soil as a bottom liner of landfill	pp. 311-312	Choton M. Mahmud and Muhammed Alamgir
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11:16~11:28	S 1418	Applications of Fiber Reinforced Polymer Composites (FRP) in Civil Engineering	pp. 209-210	Subrata Chandra Das and Md. Eanamul Haque Nizam
11:28~11:40	S 1415	Test of masonry walls for different strengthening techniques	pp. 211-212	Ataur Rahman and Tamon Ueda
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11:52~12:04	S 1431	Determination of compressive strength of mortar due to filler effect of Pozzolans	pp. 215-216	M. N. N. Khan, M. Jamil, A. B. M. A. Kaish and M.F.M. Zain
12:04~12:16	S 1433	Performance of recycled waste concrete and its applicability in construction industry	pp. 217-218	Fatima Naznin, Mossa. Farhana Jesmin, Muhammad Harunur Rashid and Md. Mafuzur Rahaman
12:16~12:28	S 1408	Study on reinforcement corrosion in high performance concrete under pre-loading condition	pp. 219-220	Abu Zakir Morshed, Hasan Ahmed Kazmee and Mohammad Zahidul Islam
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14:42~14:54	E 1452	Study on water purification by Solar Energy	pp. 077-078	Barnali Biswas, Md. Saiful Islam and Papia Biswas
14:54~15:06	E 1466	Greenhouse gas emission scenario of existing electricity generation technologies in Dhaka city	pp. 079-080	Karishma Niloy Kibria
15:06~15:18	E 1451	Doehlert design experiment to optimize pH for removal of turbidity from surface water by electro coagulation	pp. 081-082	Chanchal Majumder
15:18~15:30	E 1437	A review study on use of constructed wetlands for wastewater treatment and water recycling—exploring possibility in Bangladesh	pp. 083-084	Md. Shohidul Islam, Md. Matiur Rahman Ratan, and Md. Showkat Osman
15:30~15:42	E 1438	Removal of color from wastewater using locally available charcoal	pp. 085-086	Md. Niamul Bari, Mst. Sulekha Khatun, H.M. Rasel and Fatema Khatun
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15:30~15:42	W 1427	Possibilities of successful coastal flexible revetment technology against riverbank failure	pp. 157-158	Aysha Akter
15:42~15:54	W 1460	Development of cyclone hazard maps with effect of climate change scenario for Barguna Sarda Upazila, Bangladesh	pp. 159-160	Sadequr Rahman Bhuiyan, Abdullah Al Baky, M Mozzammel Hoque, M. Shah Alam Khan, Tarekul Islam, Mashfiqus Salehin, Shahjahan Monda and AKM Saiful Islam
15:54~16:06	W 1471	Response of protective vegetation due to wave loading	pp. 161-162	Mirza Ahammad Sharif and Iftekhar Anam
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17:28~17:40	G 1411	Finite element based design of support system for the deep excavation of an impounding reservoir: A case study	pp. 321-322	Md. Rokonuzzaman, K.M. Bipul Shahriar and Md. Kamrul Ahsan
17:40~17:52	G 1412	Study on the mechanical behavior of soft soil reinforced with coir fiber	pp. 323-324	Kazi Ehsanul Moin and Grytan Sarkar
17:52~18:04	G 1420	Pile capacity in Khulna sub-soil	pp. 325-326	Muhaimenul Alam Fiaz, Masum Shaikh and Md. Keramat Ali Molla
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17:28~17:40	S 1443	Web crippling strength of single web profiled steel sheet subjected to Interior One Flange (IOF) loading	pp. 229-230	S. M. Zahurul Islam and Md. Habibur Rahman
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19:06~19:18	S 1424	Effects of construction sequence on a continuous bridge	pp. 243-244	H. M. I. Mahmud, S.M. S. Mahmud, M. A. I. Omar and Monaz Ahmed Noor
19:18~19:30	S 1429	Strengthening technique of reinforce concrete column for one storey vertical extension: a case study of civil engineering building, kuet, Bangladesh	pp. 245-246	Shariful Islam, Mir Abdul Kuddus and Md. Iftekar Alam
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18:54~19:06	S 1422	Computer aided analysis and design of a Counterfort retaining wall	pp. 333-334	Md. Shams Adnan and Md. Keramat Ali Molla
19:06~19:18	S 1401	Chloride absorption potentiality of resin mixed cement concrete exposed to marine environment	pp. 335-336	Mohammad Najmol Haque , Hiroshi Mutsuyoshi and Osamu Sanada
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CONTENTS (Environmental Engineering)

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AVAILABLE STUDIES ON GROUNDWATER FOR DRINKING PURPOSE IN KHULNA CITY: A REVIEW

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ABSTRACT

The scarcity of water has been augmented gradually in Khulna City owing to increasing resettlement from the surrounding districts, for rapid urbanisation and industrialisation with lack of parallel growth in necessary water supply infrastructure. This paper focused on the potentiality of the aquifer recharge and the independence of the aquifers by analyzing most of previous studies available on geological, hydro-chemical, and hydrodynamic parameter of the aquifers. The estimation of annual ground water recharges in the most potential aquifer has been performed. Data on lithologs, static water level, electrical conductivity, temperature, pH, arsenic, and pumping tests in different aquifers have used in the previous studies. Most of the initial studies found that the natural recharge is very limited and in general, groundwater abstraction in Khulna can be considered as a 'mining' system of fresh groundwater. However, the final report of LGED (2005a,b; 2009) found several new findings. The important findings are: For highest static water level in the month of October, usable recharge has been obtained as about 40,470 m^3/day which is slightly more than the average abstraction rate 40,000 m^3/day. While for lowest static water level in the month of April, usable recharge has been obtained as about 36,070 m^3/day. During four years of monitoring from 2005 to 2009, the total declination of static water level (0.63 m) has not affected the total annual recharge of the deep aquifer. This observation indicates the potentiality of the aquifer for farther room of some possible additional abstraction. It is true that the complete dependency on ground water for Khulna City may not sustainable for long with the present aquifer system, alternative solution should be find out in near future.

Keywords: Fresh water, Salinity, Intrusion, Aquifer, Recharge, Temperature, KWASA.

1. INTRODUCTION

Khulna is the third largest City of Bangladesh, located at south western part of the country and approximately one hundred km north of Bay of Bengal. The City is situated in the south west part of Bangladesh and lies in the delta of the Ganges. The City itself stretches 15 km along the River Bhairab and Rupsha, covering the area of approximate 46 sq. km. having population around 1 million. The City is surrounded by lots of industries it has plenty of importance for its geographical, political, historical and financial reasons. The scarcity of water has been augmented gradually owing to increasing resettlement from the surrounding districts, for rapid urbanisation and industrialisation with lack of parallel growth in necessary water supply infrastructure. The water supply system of Khulna dates back to 1921 with a small surface water treatment plant of capacity 0.56 Million liters per day (MLD). The water supply system was expanded in 1960 by the Department of Public Health Engineering (DPHE). They have opened a new era towards ground water through installing few production tube wells in shallow aquifer. Khulna Water Supply and Sewerage Authority (KWASA) had been established on Mach of 2008. Before formation of KWASA the water supply of Khulna City was under the responsibility of Khulna City Corporation (KCC). KWASA is the third WASA in the country, following the Dhaka WASA and the Chittagong WASA. The current said production capacity of KWASA is only 25 million liters per day (MLD) through 54 production tube wells against the theoretical projected demand of 240 MLD. Nevertheless quality of water is also an issue due to wide spread arsenic contamination and salinity of the ground water. Several studies have been conducted in the past to assess the quantity of available ground water resources and find out dependable water source for the Khulna water supply system. The Department of Public Health Engineering (DPHE) and the Local Government Engineering Department (LGED) has been conducted few studies in the past to for this purpose. In a significant investigation by DPHE (Rus. 1985), it was found that Khulna City has two aquifers, shallow and deep, and opined that deep aquifer of Khulna is in mining state. After handed over the water supply system by DPHE to KCC in 1997, LGED under the frame work of Municipal Services Project (MSP) has carried out a feasibility study by Mott MacDonald Ltd (1997) to prepare an action plan for further expansion of the City water supply system. The study found that the static water level in deep aquifer is within few meters of the ground surface since starting its abstraction in 1960. This observation suggested that there might have some replenishment in the deep aquifer. This observation also suggested for further study in the area. As a result LGED (2003) has taken up another study called "Groundwater Resources and Hydro-geological Investigations in and around Khulna City (GWRHI)". Under the study they have established a broad groundwater monitoring network with a view to perform a modeling exercise for the aquifers. The project covered around 1000 sq km area where 73 different types of wells/borings have been installed for necessary investigation among them 21 monitoring tube wells has been installed in deep aquifer. Although after March 2005 no monitoring works has been conducted by any organization. Local Government Engineering Department (LGED) conducted a study in 2005, identified the presence of three different aquifers. This study also determined that these aquifers of Khulna are continuously recharging by the rainfall in the upstream. Nevertheless the upper shallow and shallow aquifer is highly contaminated by saline and upper shallow aquifer contained highly contaminated of arsenic. The deep aquifer in the central and northern part of the City contains good quality potable water with no arsenic contamination but this aquifer is surrounded by highly saline water in the south, south-east part of the City. This paper summarized the findings of the related studies the available on geological, hydro-chemical, and hydrodynamic parameter of the aquifers in Khulna city with focusing on the potentiality of the aquifer recharge and the independence of the aquifers due to fluctuation of static water levels.

2. METHODOLOGY

2.1 Collection of Study Reports

Most of the research reports, studies and literature have been collected from reliable sources including KCC, DPHE and LGED. The existence, characteristics, number of aquifers and the independence of the aquifers were analyzed by reviewing all pervious groundwater related studies the available geological, hydro-chemical, and hydrodynamic parameter of the aquifers in Khulna city.

2.2 Study Area

The study area for most of the was centered on core city area and gradually expanded for the new edition of the study. The final form of study area selected for the report LGED (2009) was around 470 sq km comprising KCC area (KCC area: 45 sq km) and its surroundings (Figure 1). Data has been collected from this extended area in a view to see the impact taking place into the KCC area. The City has a population of over one million. The topography is flat (only few meters above the mean sea level), slopes gently to southeast towards the Bay of Bengal. Previous studies reveal that groundwater is available in and around Khulna City in the shallow and deep aquifers. Water quality varies depending on the proximity to the rivers, level of confinement, depth below ground surface, and natural conditions. Previous literature related to Khulna reveals that the difference of opinion is persisting regarding the number of aquifers. According to DPHE (1985), shallow aquifers are unconfined or semi-confined. The thickness of the shallow aquifers near Khulna is sometimes difficult to ascertain, but typically varies from 0 to 100 meters. Thick silty clay layer, with or without saline groundwater, usually separate the shallow and deep aquifers. Currently, the primary source of water supply is from wells constructed into the deep aquifers, ranges between 200 and 300 m depths. BRGM-ANTEA-ARMCO (LGED, 2005a,b) classified that Khulna has three aquifers and these are upper shallow 50-75m, shallow 100-150m, and deep 200-300m thick aquifers.

3. REVIEW ANALYSIS AND DISCUSSION

3.1 Historical Overview

Several studies have been organized in and around Khulna City to provide supply water to the City dwellers. Only the resent studies recognized the complex hydro-geological characteristics of the aquifers along with problems developed due to saline water intrusion both in surface and groundwater sources. Increased water demand evolved for ever increasing population, irrigation and industrial development and complex characteristics of the coastal region. The first study in coastal zone held in 1979 by UNDP/ UNDTCD to characterize the regional nature of the aquifers. Later in 1989, under the framework of DPHE water supply and sanitation projects, a study was conducted in KCC for well monitoring and regeneration. However, the availability of those reports could not be ensured from the relevant organizations. Recent available studies have been described below.

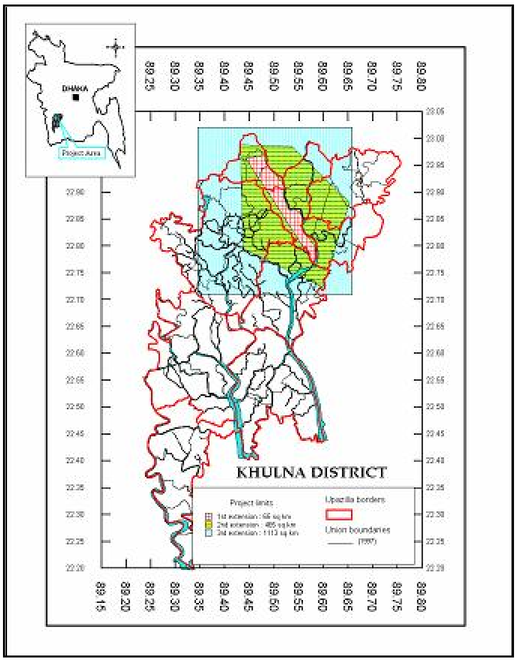

Figure 1: The final form of study area selected for the report LGED (2009).

3.2 Feasibility Study for Khulna Water Supply

Groundwater investigation survey was conducted by IWACO (DPHE, 1980) in order to find out the favourable area in Khulna district. Apart from resources in and around the City this survey yielded favourable groundwater abstraction areas in the Khulna district, at distances ranging from 10-40 km from the town. In the study the potential capacity on fresh, exploitable, groundwater in Khulna City and district surroundings was calculated as 820 million cubic meters. These calculations were based on horizontal groundwater flow in a confined aquifer system bounded in the west by an impermeable clay barrier, so that the pumped aquifer in the town was thought to be mainly recharged by (saline/brackish) water from the east. The location of the clay barrier in west and the salinity distribution were only roughly known. Particularly information on the deep formations beneath the pumped aquifer zone was lacking. The supposed continuity and impermeability of the separating clay layer between the shallow and deep aquifer was based on a limited number of drillings.

3.3 Geohydrological Investigations in Khulna

In view of expansion of the groundwater based urban drinking water supply in Khulna, geo-hydrological investigations have been carried out by J.S.Rus (1985). The investigations were implemented within the framework of the DPHE Water Supply and Sanitation Projects. These projects were a part of the Netherlands-Bangladesh Development Co-operation Program. The period in which the investigations were carried out was from July 1983 to September 1984. The investigations had been a continuation of the regional groundwater resources survey, carried out by IWACO in 1979-1980, as a part of the feasibility study for the Khulna water survey. With respect to the geological build-up as well as the groundwater quality (salinity), the hydrogeological situation in the Khulna area is rather complex. Groundwater in Khulna Town is abstracted from a deep aquifer system at a depth of 150-250m. This fresh water bearing aquifer is separated from a brackish to saline aquifer by a clay zone, which gradually increases in thickness from east to west. The deep fresh water zone in the town is laterally bounded by brackish water in the east and clay barriers in the north and west. The hydrogeological conditions at great depth (> 300m) are not well known during the study period. The main objective of the investigations was to identify the possible groundwater resources in Khulna Town and to locate favourable zones for new wells and well fields. The major findings of the investigations were;

3.3.1 Hydrogeological Context:

Fresh groundwater in Khulna is limited to the deep aquifer in the relatively narrow north – south belt, which is confined between brackish groundwater east of it and thick clay deposits west of it, while a clay layer separates the aquifer from a brackish shallow aquifer. The most favourable areas for groundwater abstraction within this zone are found in the southern part of Khulna and south of Khulna (Gollamari – Bagmara). In the northern and central part of the town areas of limited possibilities are encountered as well (Mujgunni – Boyra, Boyra – South). Mainly in the central and northern parts of the town no distinct lower boundary of the deep aquifer is encountered. Some areas in the central part of the town (Ferrrighat – Pollimangol), which produce relatively warm water, are probably underline by sandy zones of great thickness. In the southern part of Khulna the presence of lower confining layers is more common. Nevertheless the deep aquifer system in Khulna probably extends to great depth. The separating clay layer between the deep and shallow aquifer is encountered in all deep boreholes in the western part of the town. The clay layer, which contains fresh water, declines in thickness to the east. Only in the eastern part of the town short circuit groundwater flow between shallow and the deep groundwater may be present. It was found that the deep aquifer has a considerable thickness and extends beyond the present boring depth (~350m). Regarding the upper boundary conditions of the aquifer zone the investigations have proved the existence of a continuous separating clay layer, between the shallow and the deep aquifer in the western part of the town. The layer decreases in thickness in all the west – east geological cross sections in the town. As a consequence infiltration of saline/brackish water from the shallow aquifer is only to be expected in the eastern part of the town near the River Bhairab – Rupsha.

3.3.2 Recharge of the Aquifer:

The natural recharge is, in comparison to groundwater abstraction, very limited. Only in some 'leaky hot' areas natural recharge from great depth is likely to occur. Therefore, in general, groundwater abstraction in Khulna can be considered as a 'mining' system of fresh groundwater. The investigations also point to a very limited natural recharge of fresh groundwater, which is only possible from the deep formation. A very slow natural upward groundwater movement is also supported by the results of the isotope study. Regarding the whole Khulna town region, however, the natural recharge can be neglected in comparison to the preset and future groundwater abstraction. Therefore groundwater abstraction in Khulna is mainly a mining process of fresh groundwater resources. In order to obtain more accurate data on the extension of the town's deep aquifer and the salinity distribution the present program was initiated by the execution of 15 reconnaissance drillings, mainly west and south of the town. A second series of 15 test drillings, most of them closed to groundwater pumping stations, was performed to get a better insight in the groundwater flow system. In most of the drillings resistively loggings were carried out. The resistively logs were very useful to determine the vertical salinity distribution. Groundwater level measurements as well as detailed pumping tests (aquifer tests) were carried out to obtain more information on the transmissivity and thickness of the deep aquifer. In addition to the geo-electrical survey of the Feasibility Study another three geo-electrical soundings were realized for a better estimate of the salinity at great depth. Isotopic analysis were applied to determine the age of the water, whereas an extensive groundwater temperature survey, mainly by temperature logging, was realized to get a better insight in the groundwater flow conditions. From the investigations (pumping tests, groundwater level surveys and groundwater temperature measurements) it was found that the deep aquifer has a considerable thickness and extends beyond the present boring depth (350m). The investigations also point to a very limited natural recharge of fresh groundwater, which is only possible from the deep formations. A very slow natural upward groundwater movement is also supported by the results of the isotope study. The C-14 analysis of the pumped deep groundwater indicates an age of the water of about 10,000 year. Regarding the whole Khulna Town region, however, the natural recharge can be neglected in comparison to the present and future groundwater abstraction. According to this study, the groundwater abstraction in Khulna is mainly a mining process of the fresh groundwater resources. The most suitable and safe zone for groundwater abstraction was to be found in the south-western part of Khulna and south-west of Khulna; the Gollamari-Bagmara area. Two other areas of fewer possibilities are situated in the northern and central part of Khulna: Mujguni-Boyra and Boyra-South. The deep, fresh water bearing, aquifer in the Gollamari-Bagmara area has a large lateral extension, while the salinity is rather low (< 100 mg/l chlorides) and constant. In the study the potential capacity on fresh, exploitable, groundwater in Khulna City and district surroundings was calculated as 645 million cubic meters. The study also forecasted that after 30 years of continuous abstraction at the rate of 59,000 m^3/day, the usable groundwater will be ceased.

3.4 Final feasibility report, Khulna water supply expansion component (LGED 2005a,b)

LGED under the framework of Municipal Services Project has carried out a feasibility study by Mott MacDonald (1997) to prepare an action plan for further expansion of water supply system for the City dwellers. They reviewed previous works on groundwater resources in Khulna and mainly emphasized the groundwater model study done in Geohydrologial Investigation in Khulna by DPHE (1985). Apart from that they collected data from KCC DTWs. They concluded that the static water level in deep aquifer is still within few meters of the ground surface since starting its abstraction in 1960s. This observation suggested that there might have some replenishment in the deep aquifer. This observation by the feasibility study instigated further detailed study using mathematical modelling.

3.4.1 Groundwater Resources and Hydrogeological Investigation in and around Khulna City

A consortium (BRGM-ANTEA-ARMCO) conducted this study under the framework of Municipal Services Project (MSP) of LGED (LGED, 2005a,b). The duration of the investigation was 30 months (two hydrological cycles) starting from December 2002 to May 2005. They obsered that Khulna has three aquifers system as upper (50-75m), shallow (100-150 m) and deep (200-300m). The reservoir rocks of aquifers are composed by sand (course to fine) with clay layers at top and bottom. Modelling exercise conducted only for the deep aquifer in the project area. Hydraulic conductivity is set to 5×10^{-4}m/s and the model area is considered 100x100 km. In 70% area the thickness of the aquifer is considered 190m and for the rest 65m. The main conclusions obtained from the modelling task of the project are:

- The deep aquifer is able to sustain the present day level of abstractions for a fairly long time span. However, due to quality deterioration the fresh water resources will be decreased at least by a half in 2030;
- According to the modelling results the deep aquifer can not sustain any increase in groundwater draft. This would accelerate the pollution of fresh water resources by inflowing highly-mineralized water;
- Also, in case of increasing groundwater abstractions, due to increased draw-down the risk of salt water intrusion from the south may increase considerably, although at present no reliable data allowed to determine the distance of the salt / fresh water interface south of Khulna.

3.4.2 Establishment of Monitoring Network

Under the Groundwater Resources and Hydrogeological Investigations in and Around Khulna City the static water level of each well in the study area has been connected to a mean sea level (MSL) datum. All the locations of the monitoring wells and their respective levelling data are taken into the GIS map by MapInfo software. Thus the monitoring parameters such as static water level, electrical conductivity, temperature, arsenic and pH could be identified separately for each location. The monitoring wells of deep, shallow and upper aquifers are shown in Figures 2 and Figure 3 (shows the combined locations of the monitoring wells installed for three different aquifers).

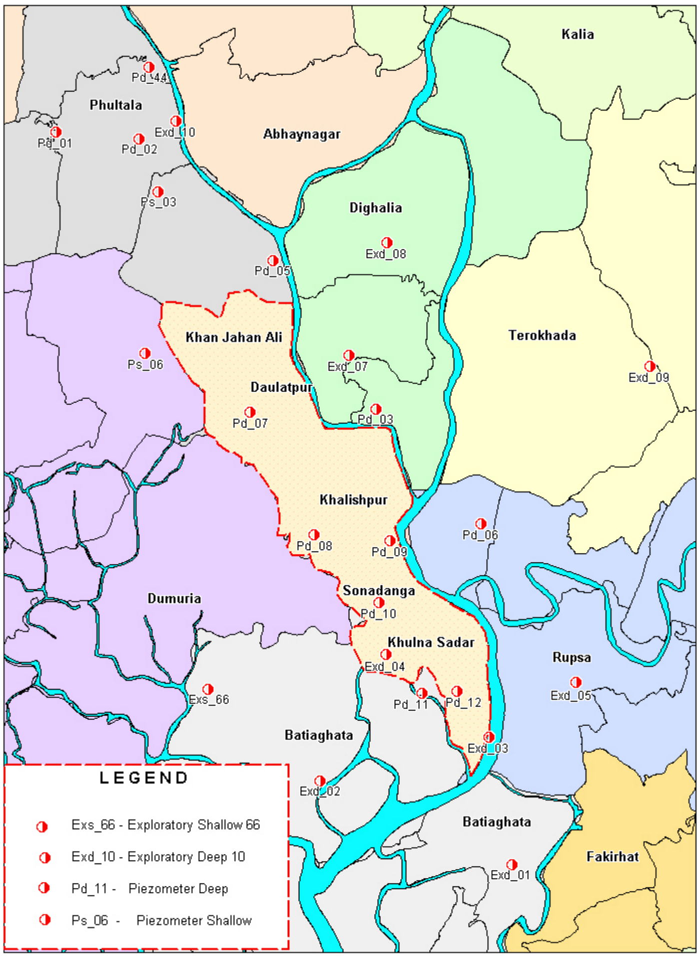

Figure 2: Monitoring Network Established by GWRHI (Deep & Shallow Aquifer)

3.4.3 Geological & Paleo- Geological Studies

Preliminary findings concerning many aspects of groundwater occurrence and yield prospects can be drawn from geomorphology and geology of the area and structure of the formations. The type of details required for geological prospecting differs from one rock type to another. Knowledge of stratigraphy and structural geology is useful in understanding the order of superposition of various lithological units and their continuity. Geomorphologically Bangladesh has been divided in to 20 units. Two of these units have been observed in the Khulna district; they are the Gopalganj-Khulna Peat Basins and the Ganges Tidal Floodplain. Their characteristics are: a) a tectonic depression with a thick deposit of a peat (up to 5 meters) covered by clay and by mostly silt and calcareous sediments and b) a flat area crisscrossed by innumerable tidal bays and streams where the sediments are mainly non calcareous clay but more silty in the east with a buried peat layer in the west. The main Paleo- geological characteristics of the study area are: the fluctuation of the mean sea level and the migration of the rivers. Concerning the mean sea level fluctuations, it has been demonstrated that, the actual level is at least 150m above levels that occurred in the past. The Quaternary study points out an important West to East migration of all rivers in this area.

3.4.4 Physico-Chemical Study of the Aquifers

Determination of the physical, bacteriological and chemical quality of water is essential for assessing the suitability of water for various purposes like drinking, domestic use, industries and agriculture. The physical quality of water includes colour, odour, taste, turbidity and temperature.

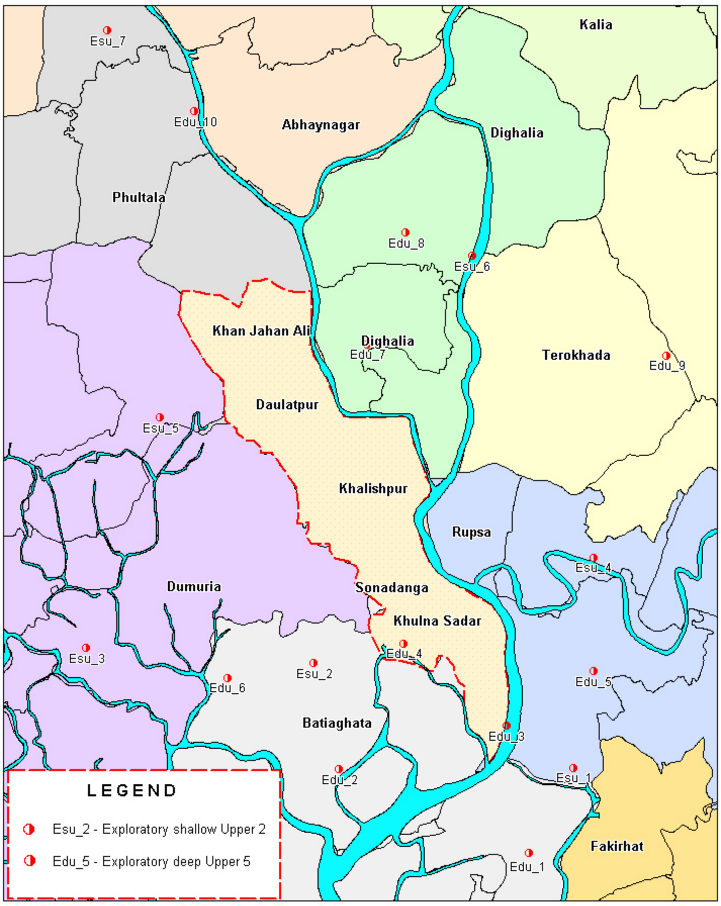

Figure: 3.2 Monitoring Network Established by GWRHI (Upper Shallow Aquifer)

Water, being an excellent solvent, acquires soluble products from the air and rocks it comes into contact with. Collection and chemical analysis of water samples from surface water bodies, wells and springs form an integral part of hydrogeological studies. Selected physico-chemical parameters of waters namely: Electrical Conductivity, pH, Temperature were measured systematically during whole monitoring sessions. Their study in high and low static water level periods shows not only their evolution but also determine the recharge area, qualitatively, the favourable area and sometime the origin of the contamination sources. In general, conductivity is a value that represents how easily electrical charges can be transported through a conductor. Resistivity is the reciprocal of conductivity. In water quality determinations, conductivity, defined as the conductance of a cube of a substance one centimetre on a side, is reported in mhos/cm. Conductivity is the most readily measurable, and also most useful single indirect chemical determination that can be made on water. For convenience, conductivity is expressed in micromhos/cm, which is equal to one million times the value given in mhos/cm. Conductivities for a given concentration of different salts is not the same. As natural waters commonly contain several salts in solution, no definite relation exists between the conductance and dissolved solids.

Table 1: Conductivity of Various Aqueous Solutions (at 25 ^{0}C)

Type of Water	Conductivity (µS/cm)
Pure Water	0.05
Distilled Water	0.05 to 1
Drinking Water	500 to 1,000
Brackish Water	1,000 to 8,000
Ocean Water	53,000

Source: - Eutech Instruments Pte. Ltd. 1997

3.4.5 Diurnal Auto - Fluctuation of Water Level

An important fluctuation of water level has been observed under static conditions, on the piezometric level. Such auto-fluctuations might be origin of a lot of observed anomalies during the groundwater flow studies (iso-piezometric map) as well as during the evaluation of the sustainable reserve of aquifers. To identify the extent of this effect on the area, in first step, few TW which are located not far from the river Bhairab, have been continuously surveyed for a period of 36 hours. To widen this study all TW were monitored continuously for 24 hours and Bhairab River for one month.

3.4.6 Litho-Stratigraphical Studies

3.4.6.1 Lithology

Geological and geophysical surveys are sometimes not adequate to interpret subsurface hydrogeological conditions. Even if surveys indicate the presence of aquifers, the productivity and quality of water needs to be evaluated. Aquifer and reservoir dimensions and boundaries, lithofacies variations and structure of formations are also to be investigated on a regional scale for a planned ground water development. Thus, it may be necessary to know lithological (and some times hydrogeological) variations by drilling. The drilling samples of the 63 new boring sites have been analysed and computerised by using different softwares (including GIS, database and ACTIF).

3.4.6.2 Sub-surface Structural Geology (Reservoir Rock Structure)

Knowledge of stratigraphical and structural geology is useful in understanding the order of superposition of various lithological units and their continuity. The correlation of the fifteen (15) litho-stratigraphical cross sections has been prepared base on the 63 exploratory drilling under GWRHI and the previous available lithologs. These litho-stratigraphical cross sections have permitted to study south-north and east-west lithological evolution of the encountered groundwater bearing layers.

3.4.7 Summary of the Study

In order to assess the situation of the deep aquifer of Khulna during 2009 two special ground water monitoring session, on the MSP monitoring network, has been conducted under the financial assistance of Asian Development Bank (KWASA 2009). It is found that there is no indication of increase of average temperature neither in KWASA production well nor in MSP monitoring well since 2005. It is also observed that the average temperature of KWASA production wells in the central part of the City is much higher than those in other part of the study area. There is no increase in the average EC value for the MSP monitoring wells since 2005. The average SWL in the project area was (-) 1.42 m from mean sea level (MSL) in 2005 and in 2009 it becomes (-) 2.06 m from MSL thus the net average declination of SWL for the whole project area becomes 0.63 m in last five years. Nevertheless the net single point maximum declination of SWL recorded is 1.00 m (Exd_04). This mass of declination of the SWL has affected neither EC nor Temperature though the average temperature is higher than the normal range. The fluctuating water heights from the monitoring wells of deep, shallow and upper aquifers are 2.13 m, 2.26 m and 1.97 m, respectively. Transmissivity and storativity have been evaluated in the study area from the collected pumping test data. It has been observed that the transmissivity values vary between 1.3×10^{-3} m^2/sec and 7.1×10^{-2} m^2/sec and the storage coefficient varying between 9.7×10^{-4} and 4.7×10^{-3}.

For highest static water level in the month of October, usable recharge has been obtained as about 40,470 m^3/day which is slightly more than the average abstraction rate 40,000 m^3/day. While for lowest static water level in the month of April, usable recharge has been obtained as about 36,070 m^3/day. During four years from 2005 to 2009, the total declination of SWL (0.63 m) has not affected the total annual recharge of the deep aquifer. This observation indicates the potentiality of the aquifer for farther room of some possible additional abstraction. It is true that the complete dependency on ground water for Khulna City may not sustainable for long with the present aquifer system, alternative solution should be find out in near future.

Since the fresh water resource of the deep aquifer is limited and cannot sustain substantial increase in abstraction rates, an additional volume of drinking water may be derived from blending of water of different sources. The Electrical Conductivity of the deep aquifer near and South of the City centre is very low. There might have few solutions to mitigate the water crisis of the City; i) if this water is mixed with a small quantity of moderately mineralised water from the shallow aquifer the overall quality will be acceptable and the overall water production can be increased by 25% or more. It is reminded that the water quality of the shallow aquifer is good (and arsenic free) except for the high salt content also with some iron. Iron of the shallow aquifer might be removed by simple aeration and filtration technique. ii) preserve and restrict the abstraction from deep aquifer water for the period of low salinity in the surface water (7-8 months time in the Bhairab- Rupsha River) and supply the ground water for the period of high saline in the surface water (February – June). iii) impose control over the industrial abstraction from the ground water and provide them alternate water by the surface for throughout the year.

4. CONCLUSIONS

According to geo-hydrological investigations carried out by J.S.Rus (1985), groundwater in Khulna Town is abstracted from a deep aquifer system at a depth of 150-250m. This fresh water bearing aquifer is separated from a brackish to saline aquifer by a clay zone, which gradually increases in thickness from east to west. The deep fresh water zone in the town is laterally bounded by brackish water in the east and clay barriers in the north and west. The natural recharge is, in comparison to groundwater abstraction, very limited. At that time it was analyzed that, in general, groundwater abstraction in Khulna can be considered as a 'mining' system of fresh groundwater. According to LGED (2005), for highest static water level in the month of October, usable recharge has been obtained as about 40,470 m^3/day which is slightly more than the average abstraction rate 40,000 m^3/day. While for lowest static water level in the month of April, usable recharge has been obtained as about 36,070 m^3/day. During four years from 2005 to 2009, the total declination of SWL (0.63 m) has not affected the total annual recharge of the deep aquifer. This observation indicates the potentiality of the aquifer for farther room of some possible additional abstraction. It is true that the complete dependency on ground water for Khulna City may not sustainable for long with the present aquifer system, alternative solution should be find out in near future.

REFERENCES

KWASA (2009). Report on Two Special Ground Water Monitoring Session on the MSP Monitoring Network. Municipal Services Project of LGED, Khulna.

LGED (2003). Municipal Services Project of LGED. Bi-monthly reports; *Ground water Resources and hydrogeological Investigations in and Around Khulna City*. Bi-monthly Report –II and Bi-monthly Report – II.

LGED (2005a) Municipal Services Project of LGED. *Ground water Resources and hydrogeological Investigations in and Around Khulna City*. Volume- 1 (Main Report).

LGED (2005b) Municipal Services Project of LGED. *Ground water Resources and hydrogeological Investigations in and Around Khulna City*. Volume- 2 (Annex- I) - Volume – 7 (Annex- VIII).

MacDonald. M. (1997). Municipal Services Project of LGED. Feasibility Report; *Khulna Water Supply Expansion Component*. Volume -I. Main Report.

MacDonald. M. (1997). Municipal Services Project of LGED. Feasibility Report; *Khulna Water Supply Expansion Component*. Volume -II: Appendices.

Rus J.S., (1985). *Water Supply and Sanitation Projects* (Netherlands -Bangladesh Development Cooperation Program) IWACO study for DPHE

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EVALUATING THE EXISTING CONDITION OF CLINICAL WASTE MANAGEMENT IN KHULNA CITY AND ITS POSSIBLE HEALTH HAZARDS

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ABSTRACT

A huge amount of clinical waste is generated everyday in various cities of the countries of the world due to urbanization and people growth. Thus the clinical waste management has become one of the crucial environmental concerns everywhere. This study mainly shows the existing clinical waste management and waste generation rate in Khulna city. The methodology of this project was comprised of questionnaire survey and interviews with the authorities and personnel of various health centers. The total waste generation in Khulna city is about 520 ton/day among which the amount of clinical waste is 2.5 ton/day. KCC authority and Prodipan take part in off-site transportation of clinical waste from various health centers to dispose at Rajbandh about 8 kilometers to the south of Khulna city. Even though Prodipan expands its hand in managing of clinical waste, nevertheless the condition of maximum health centers is dangerous due to lack of sufficient care, workers and money. That is why proper segregation, collection, transportation, final disposal, public awareness and training of staff are obligatory for sound medical waste management and all metropolitan cities in Bangladesh should be more improved in their respective clinical waste management practices.

Keywords: Clinical wastes, disposal practice, health facilities and waste generation, sustainable management plan.

1. INTRODUCTION

Hospital is a service-oriented residential establishment that provides medical care facilities comprising of rehabilitative, diagnostic, observational, and therapeutic services for persons suffering from or suspected to be suffering from any kind of disease or injury (Rahman et al., 1999). Nowadays, the study of clinical waste has become a matter of major concern due to the socio-economic development, urbanization as well as growth of population in most cities of developing countries in the world. The clinical waste management in developing countries is still poor and disposed without adequate experience and supervision. In some other countries, hospital effluents have not been legally declared (Dutta, 1998; Kwok-Kuen, 1998). Hassan et al. (2008) reported, wastes of medical centres are serious threat to environment and needs specific treatment and management prior to its final disposal. One of the important parts of solid waste, waste generation from health care centres, is a serious problem both in the developed and developing countries. Clinical waste as a whole is being disposed with the municipal garbage (Pathak, 1998). Waste products from hospitals or clinics are not treated or destroyed properly, rather thrown into the dustbins thereby creating health hazards (A. A. Talukdar, 2009). Disposal of clinical waste without treatment causes a serious public health hazards. Infectious wastes have the potential of transmitting infectious agents to humans. Unintentional injuries may occur when the community is exposed to inadequately disposed waste (Turnberg, 1996). So a satisfactory clinical waste management system in government hospitals, clinic, diagnostic centres and pathological laboratories is severely lacking. The environmentalists are becoming very much worried because of the hazards of clinical waste as the medical facilities are increasing day by day. The clinical waste is a source of contamination to both humans and the environment which is capable of causing diseases and illnesses to people, either directly or indirectly by contaminating groundwater, soil, surface water and air.

Khulna, the biggest metropolitan city of Bangladesh, is surrounded by rivers named Rupsha and the Bhairab. Establishment of various health care centers, clinics and hospitals has made difficult for management of different waste (A. U. Jabbar, 2009). Generally, medical waste is defined as the discarded or unwanted material or garbage or solid waste which is generated from the treatment, diagnosis, or immunization of human beings or animals, in research pertaining there to, or in the production or testing of biological materials (Lee, 1989).Healthcare waste contains infectious pathogens, heavy metals, toxic chemicals and may contain substances that are genotoxic or radioactive. These substances can cause adverse health effect to humans and the environment if not handled properly. Of particular concern is the risk of infection to those who handle the waste and of the general public (Mbongwe *et al.,* 2008). The present practice of improper handling of generated clinical wastes in Khulna city is playing a contributing rule in spreading out various diseases such as diarrhea, tuberculosis, heamorrtetanus, AIDS, STD, meningitis, infection of the liver, stomach, breathing infection, infection of the reproductive organs, various skin diseases, etc. However Khulna is currently facing the impacts of improper management of clinical wastes. Often medical staffs were found to generate revenue through sale of medical waste due lack of knowledge and interest in safe waste disposal and absence of a budget to effectively implement safe waste disposal (Akter, 2000). Realizing the intensity of the problem, some NGOs have already extended their helping hand to KCC for the management of clinical waste, yet the existing management system is a threat to environment and human health (Khandaker, 1999). At present, the clinical waste is one of the vital problems in Khulna city. All cities of the countries should establish waste treatment and disposal criteria according to the national and international guidelines. This study aimed at investigating the existing clinical waste management and its possible health hazards in Khulna city and quantifying the amount of clinical waste generated from various health centers.

2. METHODOLOGY

2.1 Field Survey

The methodology for this study includes field observation and field level data collection by means of questionnaire survey and interviews in formal as well as informal ways. Data were mainly collected from the primary data sources and some from secondary. Primary data were collected by means of questionnaire survey in the field. A number of formal and informal approaches were done in order to collect data. Also interviews were conducted with the involvement of the hospital waste management authority, word master, lab-technicians, nurses and cleaners of the hospital and office personnel of clinics and diagnostic centers, KCC and NGO's. Set of questions ended up being done along with men and women linked to delivering clinical waste materials dealing with and pretreatment associated with clinic waste materials before final disposal. After collection of data, it was possessed, analyzed and interpreted by the graphical presentation to address the central issues of clinical waste management with relation to the generation of wastes. The total amount of generated waste, amount of clinical waste, generation rate and current clinical waste management in Khulna city were identified. The no. of bed, amount of general and hazardous waste, waste generation rate in 20 popular city hospitals and clinics were determined and also a idea about training and awareness in KCC area were obtained. The information of the study area was collected from the secondary data. The treatment of clinical waste at dumping site has been also proposed to improve the clinical waste management.

2.2 Study Area

Khulna, the third largest industrial and second largest port city, is located in the south-west of Bangladesh. It lies between 22.04°7′16′′ to 22.05°2′00′′ north latitude and 89.03°1′36′′ to 89.03°4′35′′ east longitude. The city has an area of 45.65 square kilometers (17.62 square miles) and has a population of near about 1.5 million. The city, however for administrative purposes, is divided into 31 wards and each ward consists of different *mahalla*s and the total number of which is 143 (Population Census, 2001). Khulna Municipality was established at 12th December in 1884, upgraded in to Municipal Corporation at 10th December in 1984 and renamed as a City Corporation at 6th August in 1990. The city is situated at a road distance of about 333 km from the capital Dhaka.The city is 4 meters above the mean sea level (MSL). It is surrounded by rivers named Rupsha and the Bhairab. From the past Khulna became glorious for the presence of the seaport named Mongla and Mangrove forest named Sundarbans. However there is an economic dominance of Khulna on national economic development and It is also established as a regional centre for trade and commerce. As a consequence the city generates a huge quantity of waste equal to about 520 tons of municipal solid waste everyday from different types of sources. The maximum portion of these wastes is organic waste and the rest is inorganic waste. Though all clinical waste management programs are performed by both government and NGO's, the practical situation of waste management of Khulna city is not much satisfactory. So the authority must give proper attention for the improvement of existing clinical waste management system in Khulna city area.

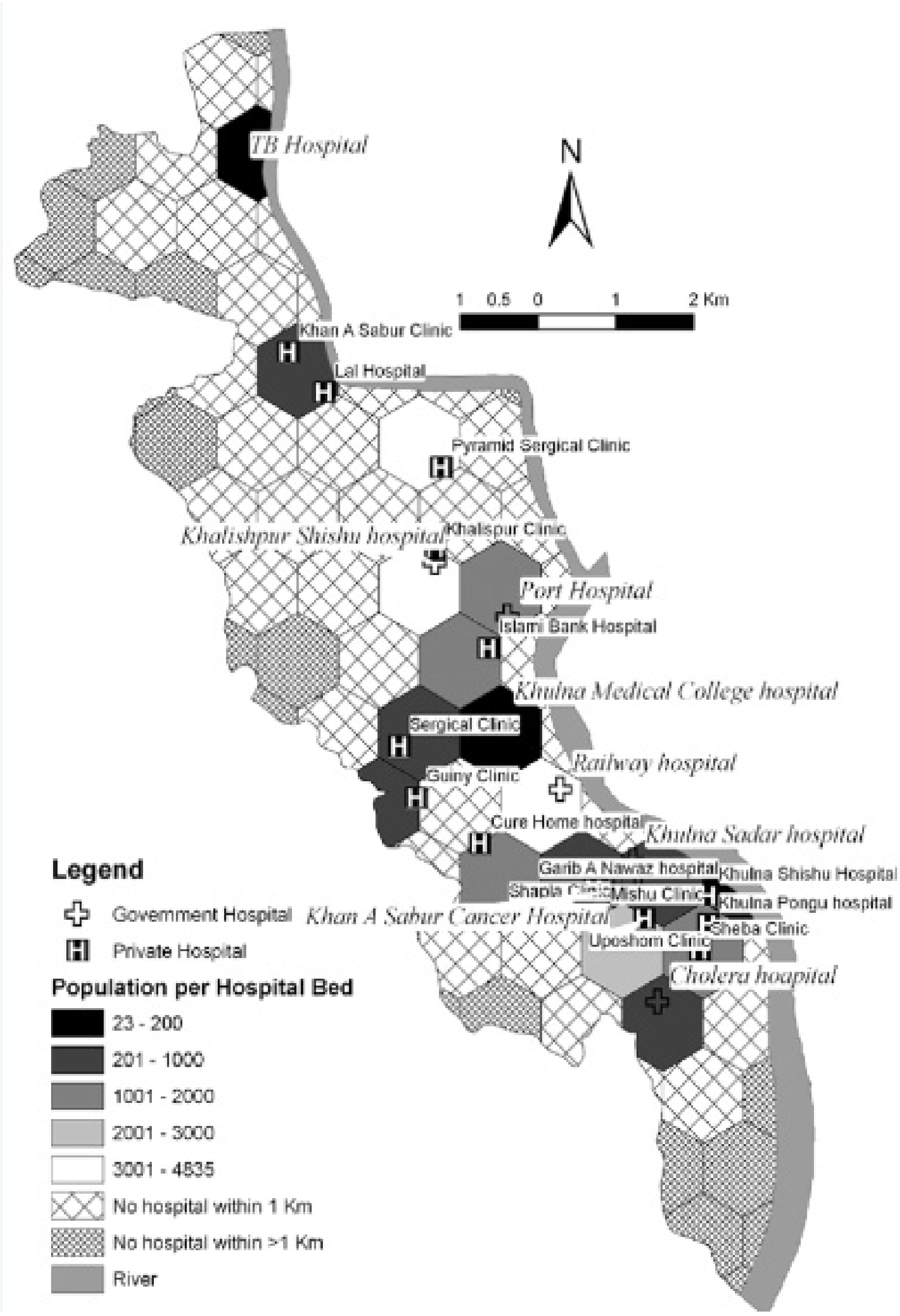

Figure 1: Position of various hospitals and clinics in KCC area

3. RESULTS AND DISCUSSION

There are 10 governmental hospitals, 15 private hospitals, 60 clinics, and 65 diagnostic centers at Khulna City Corporation area according to the report of KCC and NGO.

Clinical waste is nothing but the waste generated by hospitals, clinics, diagnostic centers, pathological laboratories. Clinical waste consists of syringes, bandages, saline bags, live vaccines, body parts, bodily fluids and also waste materials, razor-sharp small needles etc.

The following data were collected by questionnaire survey in the field from KCC, NGO's and different hospitals. Among them the data of clinical waste of some popular hospitals and clinics is given below in a tabular form:

Table 1: Waste generated from various hospitals and clinics

Name of the centre	No of bed	General waste kg/day	Hazardous waste kg/day	Total waste kg/day	Waste generation rate kg/bed/day
KMCH	500	450	150	600	1.20
KSaH	250	306	54	360	1.44
GMCH	250	229	84	313	1.25
KSH	200	114	27	141	0.71
IBH	100	49	17	66	0.66
NMH	100	37	8	45	0.45
SMH	100	9	12	21	0.21
KPH	100	21	4	25	0.25
NCH	100	24	6	30	0.30
PGH	50	14	3	17	0.34
SSANSH	50	24	4	28	0.56
BH	50	40	5	45	0.90
KH	50	13	2	15	0.30
NMSC	20	16	4	20	1.00
SHC	20	15	3	18	0.90
MSC	20	26	19	45	2.25
RMH	20	23	6	29	1.45
SMC	20	33	31	64	3.20
GNC	20	17	13	30	1.50
AAMCH	20	21	17	38	1.90

(KMCH=Khulna Medical College Hospital, KSaH=Khulna Sadar Hospital, GMCH=Gazi Medical College Hospital, KSH=Khulna Sishu Hospital, IBH=Islami Bank Hospital, NMH=Nargis Memorial Hospital, SMH=Santa Maria Hospital, KPH=Khulna Police Hospital, NCH=Navy Camp Hospital, PGH=Pongu and Gyne Hospital, SSANSH=Shahid Sheikh Abu Naser Specialized Hospital, BH=Bokkhobadhi Hospital, KH=Kara Hospital, NMSC=Nagar MatriSadan Clinic, SHC=SurjerHasi Clinic, MSC=Meri Stops Clinic, RMH=Rashida Memorial Hospital, SMC= Shamela Memorial Clinic, GNC=Garib Newaz Clinic, AAMCH=Ad-din Akij Medical College Hospital)

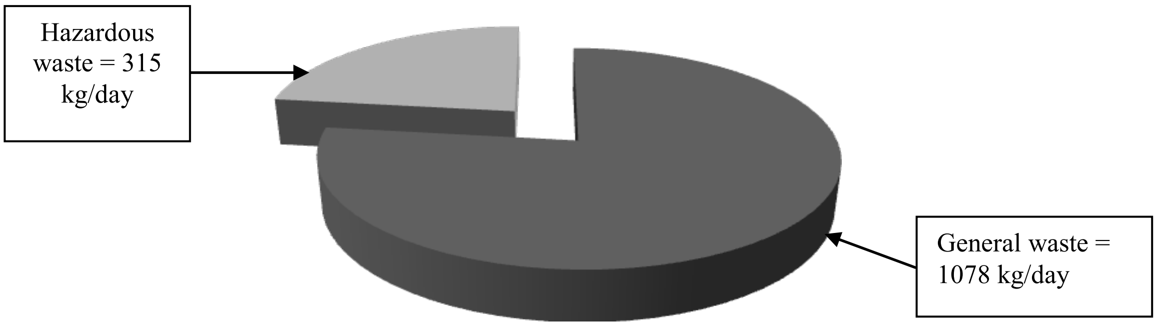

Figure 2: Amount of general and hazardous waste in various health centers

Though WHO (1999) classified clinical wastes in nine categories, here it is categorized in four types for the Khulna City hospitals.

- Infectious Waste: This includes bandages, human organs, cotton, sticking plaster, gauges, surgical gloves, tissues, body fluids or excreta, dressings, pathological waste etc.
- Plastic type waste: This includes tubes, saline bags, ampoule, vial, blood bags etc.
- Sharps: This includes all hypodermic needles and syringes, blades, scissors, ampoules, vials without content, lancelets and broken glass etc.
- General Waste: This includes food waste, cardboard, packaging, plastics, office paper, non-contaminated plastic or metal, cans or glass.

Figure 3: (a) Infectious waste (b) Sharp waste (c) Plastic waste (d) General waste

3.1 Types of Waste Generated

All the wastes have been categorized in four groups according to their composition and hazardous character. They are: infectious, sharps, plastic and general waste. A study simply by Prodipan (2012) has confirmed that 76% of organic waste and 24% of inorganic waste is generated in Khulna city corporation region. The amount of various wastes per day is given below:

Figure 4: Organic and inorganic waste pie chart

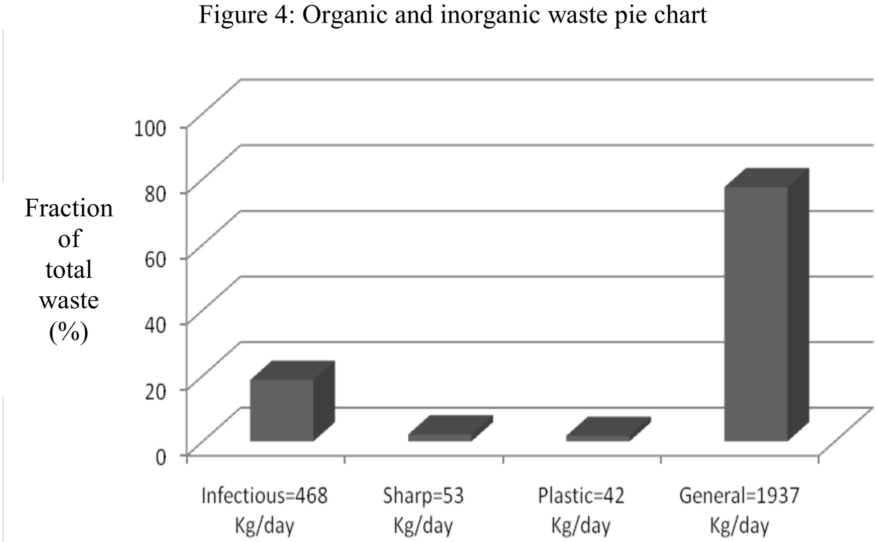

Figure 5: Column chart of various waste in KCC area

3.2 Clinical Waste Management

3.2.1 Role of Khulna City Corporation in Clinical Waste Management System

Though Khulna City Corporation (KCC) is the public body for the management of wastes in Khulna city, KCC authority does not have any special arrangement for the collection and disposal of clinical wastes because they are not obliged to do according to their ordinance. Skillful and trained manpower is essential for the proper clinical management system but there have some problems such as shortage of fund, manpower, vehicles etc.

KCC gathers all types of wastes including clinical wastes in the similar vehicle 2 times per day from public dustbins. Generally 16 pickups go for the collection of wastes at a time, although KCC provides 20 pickups along with regards to 200 wheel carts and after collection the clinical wastes are dumped together with other wastes. The pickups are used for collecting wastes only from primary dustbins. Alternatively, the wheel carts deliver wastes from the secondary dustbins to the primary dustbins.

3.2.2 Role of Private Organization in Clinical Waste Management System

Some NGO's are involved in clinical management system in Khulna city such as Prodipan, Muktir Alo etc. Prodipan a great NGO is playing a new making contributions position throughout managing Khulna city clinical waste. Prodipan took a noble initiative to start their clinical waste management service in May, 2000 with funding from Swiss development cooperation (SDC), UNDP and World Bank. Prodipan felt it when it was developing a community based solid waste management in the city. The vision of Prodipan is the developing a sustainable society for ensured standard of living and improved quality of life for the underprivileged. With beginning phase of their operation Prodipan didn't get virtually any waste treatment plant and used to dump waste casually. Not long ago Prodipan has brought some initiatives to expand its program to include clinical waste treatment before dumping and has installed its own waste treatment plant dumps waste after treating it. The activities of Prodipan are increasing day by day and more people are involved with this NGO Now Prodipan has covered about 70% of all clinical waste management in Khulna City Corporation.

Prodipan provides each hospital with minimum a set of four covered drums to dispose four types of waste separately to segregate waste at the source of generation. These four drums are notable having four different colors with regard to uncomplicated identity. The quantity of set of drums are different having how big the hospital. But basically many experts have noticed that numerous health centers have been provided with lower than four drums. A vehicle truck carrying 1.5 ton waste is used for transportation of clinical waste from the various health facilities in KCC area.

3.2.3 Final Disposal Method and Dumping Site

In the management of clinical wastes safe disposal is the most important thing. Rajbandh, about 8 kilometers to the south of Khulna city is used for dumping of the waste. KCC authority uses different open location to dump waste for land filling which increases the risk of health hazard of the local community.

Figure 6: (a) Local waste disposal site (b) Waste collecting pickup car of Prodipan

3.2.4 Potential Health Hazards due to Untreated Clinical Waste

The risk of untreated clinical waste can be higher if the clinical waste is not properly treated. The people who are involved with the clinical system in health centers are potentially at risk

- Doctors, nurses, personnel of hospital maintenance of the health care center.
- Patients in health care center or may be getting home care.
- Visitors to health care center.
- Workers who collects and transport waste.
- Workers and scavenger in waste disposal or landfill area.

Many types of injury can be such as punctured wound, laceration, cut-injury, backache due to load hauling, strain and sprain of the joint of limbs. Infectious waste contains various kinds of pathogens and organism such as bacterial, parasitic, viral, fungal infections and these pathogens causes various types of fatal diseases. There is also potential risk of typhoid, diarrhea, dysentery, HIV or AIDS, TB or throat infection, bacterial or viral diseases etc.

- Tetanus, gangrene and other wound infection, anthrax, cholera, other diarrhea diseases, enteric fever, shigellosis, plague etc. are caused by bacterial action.
- Various hepatitis, poliomyelitis, HIV-infectious, HBV, TB, STD rabies etc. are caused by viral action.
- Ameobiasis, girdiasis, malaria, ascariasis, echinococcosis, leishmaniasis, ankylomastomiasis, filariasis etc. are caused by parasitic.
- Various fungal infections like candidiasis, cryptococcoses, coccidiodomycosis etc. are caused by fungal action.

Health hazards due to clinical waste

- Drinking water can be contaminated and leachate can enter in an aquifer, surface water or drinking water system.
- Non-biodegradable antibiotics of the clinical waste may kill bacteria necessary for sewage treatment.
- Toxic pollutants release into the air if the waste is burnt at low temperature.
- Insecure and unprotected landfill may pose health hazard to the nearby people of landfill area.
- Improper disposal of clinical waste causes foul odors, favors fly feeding and contaminates both water and air.

3.2.5 Proposals for Improving the Existing Clinical Waste Management

For improving the existing clinical waste management in Khulna city area, the following flow path is suggested. In this flow path, two flow diagrams are shown. First one is for the safe management for overall clinical waste and the second one is for the management for the specific clinical waste. For overall clinical waste, the proper management procedure incorporates waste segregation, storage of waste at local dustbin, waste handling properly, collection of waste, safe transportation by transporting media and final disposal of waste at dumping site after treatment. However for particularly infectious wastes they should be dried with Ca(OCl)Cl, washed and then burned in furnace. Sharp type material should be disposed in a concrete pit. Plastic wastes are wetted, so they must be washed by Ca(OCl)Cl, cut to be unsuitable for using and then sold. General wastes are usually dumped openly. Also the degree of understanding and the level of awareness of the people about the health hazards happening due to this uneven management should be increased.

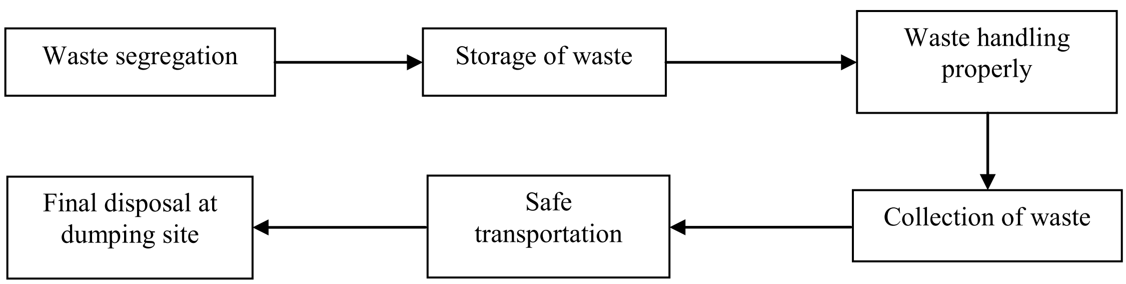

Figure 7: Flow paths for improving the total clinical waste management system

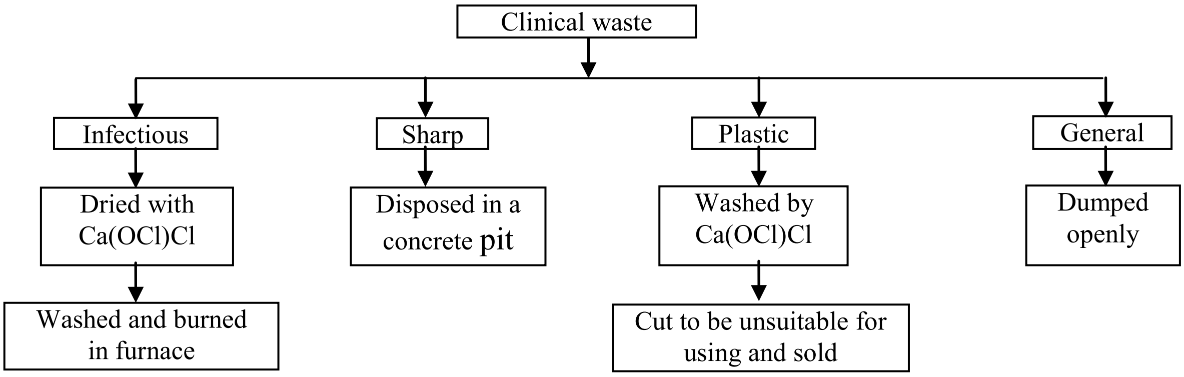

Figure 8: Flow paths for improving the management system of various clinical waste

4. CONCLUSIONS

The clinical waste management system is becoming certainly one of most critical issue all over the world. This study had been mainly executed to possess a distinct thought about the clinical waste management practices in addition to quantity the amount of total clinical waste generated and its possible health hazards in Khulna city area. The generation of hazardous waste is about 0.6 ton everyday in KCC area. About 23% of total waste is hazardous wastes which are generated everyday in Khulna area. The authorities are unwilling to take penal steps in opposition to poor convenience associated with hazardous waste materials. Thus the current method associated with clinical waste management in KCC area is neither satisfactory none ample. Since KCC doesn't supply proper interest for the collection and disposal of clinical wastes, so it should require a lot more Non-Governmental Organizations to improve the particular clinical waste materials convenience method. However the involvement associated with Community NGO Prodipan covers a lot of the clinical management system within Khulna City Corporation. From time to time many cleansers are engaged to mishandle the particular earned waste items. They just don't segregate infectious waste items coming from non-infectious waste items in addition to dispose the wastes to the open dumping site. Improper disposal of waste in landfill generates various gases such as methane (CH_4), nitrogen (N_2) and sometimes hydrogen sulfide (H_2S) and if burnt carbon di-oxide (CO_2) is released and affects green house. So hygienic in addition to affordable actions have to be used for the final disposal and treatment of the hazardous waste. Individuals need to learn concerning the undesirable consequence associated with clinical waste in environment. So general public recognition and training should be used as the key components intended for proper management of clinical waste. The authority of the clinical waste management should follow WHO guideline properly adequately. Thus, the idea becomes urgent for that KCC authority and the staff members on the medical centers to adopt the clinical waste materials supervision plan using the guidelines of World Health Organization (WHO) to attenuate the hazards of clinical waste.

REFERENCES

Akter, N. (2000). “Medical Waste Management : A Review”, Environmental Engineering Program. School of Environment, Resource and Development, Asian Institute of Technology (AIT).

Dutta, Subijoy (1998). “Best Practicable Management of Medical Waste in India”, Proceedings of National Workshop on Management of Hospital Waste, Jaipur, India.

Hassan, M.M., Ahmed, A.S., Rahman A.K. and Biswas, K.T. (2008). “Pattern of Medical Waste Management: Existing Scenario in Dhaka City’’, Bangladesh, BMC Public Health, Pages.1-10.

Khandker, F. (1999). ‘‘Clinical Waste Management in Khulna City’’, BURP Dissertation Paper, Khulna: Urban and Rural Planning Discipline, Khulna University.

Lee, C. (1989). “Environmental Engineering Dictionary”, Government Institutes, Inc., MD 20850.

Mbongwe, B., Mmereki, B.T., Magashula, A. (2008). "Healthcare waste management: current practices in selected healthcare facilities, Botswana’’. Waste Management 28 and 226–233.

Pathak, Sati. (1998). ‘‘Management of Hospital Waste: A Jaipur Scenario’’, Proceedings of National Workshop on Management of Hospital Waste, Jaipur, India, April 16 - 18, 1998.

Population Census (2001). Community Series, Zilla: Khulna. BBS, Ministry of Planning, GOB.

Rahman, M.T., Sarker, M.S.K.A., Chowdhury, M.A., Haque, K.E. and Hasan, M.M. (2003). "Solid Waste Pollution in Sylhet City”. Bangladesh J. Environ. Sci., 9: 297.

Turnberg, W. L. (1996). "Bio Hazardous Waste – Risk Assessment, Policy and Management”. USA.

WHO. (1999). “Guidelines for safe disposal of unwanted pharmaceuticals in and after emergencies” Geneva: Essential drugs and Other Medicines Department, World Health Organization.

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STUDY ON COMPOSTING OF UNCOOKED KITCHEN WASTE

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ABSTRACT

The increasing quantity of organic solid wastes is one of the major environmental problems in developing countries like Bangladesh. A large volume of organic solid wastes are produced in the residential houses of cities, towns and rural areas everyday which are needed to be treated before dumping to minimize the detrimental effects of open dumping of organic solid wastes. Paying attention to the effective disposal of organic kitchen wastes after converting as compost, the study was conducted to investigate the effects of forced and passive aeration on composting process. A total of four runs of composting were performed using composting reactors. All the wastes were mixed with selected proportions. Physical parameters like moisture content and volatile solids were determined before and after composting. The temperatures inside reactors were measured daily. The test results showed that the temperature inside reactors increased above 55 ^{0}C during composting. This temperature is very effective to destroy most of the pathogenic microorganisms. The percentage reduction in total weight, moisture content and volatile solids proceeded more with time with an average reduction value of 37.95%, 43.32% and 28.98% respectively. The compost found after the fourth run was more mature than that of third run which offer fertilizer for cultivation.

Keywords: Composting, Aeration, Moisture content, Volatile solids, Temperature.

1. INTRODUCTION

A lot of solid materials are used in all technical society. These materials are discarded as useless or unwanted after utilization. These materials produce wastes everyday in a society. The term 'Solid waste' means the heterogeneous solid or semi-solid masses of throwaway from residence and commercial activities as well as the more homogeneous accumulations of any single industrial activity. The organic solid wastes produced in a kitchen of residential houses are called kitchen wastes. Most of these wastes are thrown away without any treatment in our country, which pollutes the environment. The treatment of these wastes biologically after collection is called composting of kitchen wastes. In early times, the disposal of wastes did not pose a significant problem, as the population was small and the amount of land available for the assimilation of wastes was large. But at present the increasing quantity of solid wastes is a growing environmental problem in developing countries like Bangladesh because large volumes of solid wastes constitute an enormous public health and environmental problem such as diseases transmission by insects, odor nuisances, atmospheric and water pollution, aesthetic nuisance and economic losses. It is necessary to treat the wastes properly as treated wastes are less harmful for the environment. The process of composting can make a contribution to the solution of all these problems. Composting is the main biological process of treating the organic portion of solid wastes (Bari and Koenig 2001). It is an ancient resource recovery process practiced, though less frequently, in both developing and industrialized parts of the world. Composting is termed as the process of bacterial conversion of organic solid and semi-solid wastes into compost which can be handled, stored and transported without any adverse environmental effect, and can be used as organic manure for improvement of soil quality and fertility (Finstein, et. al., 1986). The end product remaining after bacterial activity in composting of organic wastes is called compost or humus (Haug 1993). The entire process involving both the separation and bacterial conversion of the organic solids is known as composting. Decomposition of the organic solid wastes may be accomplished either aerobically or anaerobically, depending on the availability of oxygen (Haug 1993, Tchobanoglous et al. 1993). In general, the operation of anaerobic processes is more complex than that of aerobic processes. However, anaerobic processes offer the benefit of energy recovery in the form of methane gas and thus are net energy producers. Aerobic processes, on the other hand, are net energy users because oxygen must be supplied for waste conversion, but they offer the advantage of relatively simple operation and, if properly operated, can be significantly reduced the volume of the organic portion of municipal solid waste (Tchobanoglous et al. 1993). Different types of bacteria, fungi, molds, and other living organisms are present in compostable materials (Bari 2011). However, in the process of composting, microorganisms break down organic matter and produce carbon dioxide, water, heat, and humus, the relatively stable organic end product. The heat produced causes the compost temperature to rise rapidly. As the temperature rises above about 40 °C, the mesophilic microorganisms become less competitive and are replaced by others that are thermophilic, or heat-loving. At temperature of 55 °C and above, many microorganisms that are human or plant pathogens are destroyed (Epstein, 1997). Because temperatures over about 65 °C kill many forms of microbes and limit the rate of decomposition, compost managers use aeration and mixing to keep the temperature below this point. During the thermophilic phase, high temperatures accelerate the breakdown of proteins, fats, and complex carbohydrates like cellulose and hemicelluloses, the major structural molecules in plants. As the supply of these high-energy compounds becomes exhausted, the compost temperature gradually decreases and mesophilic microorganisms once again take over for the final phase of "curing" or maturation of the remaining organic matter (Bari, 2011). There are several important parameters such as moisture content, temperature, C/N ratio, pH control, particle size, oxygen and aeration etc. which have to be in desirable conditions for efficient aerobic composting to occur at high temperatures. Again, the maturity of compost affects its successful utilization in agriculture. A number of physical, biological and chemical methods have been suggested to measure the maturity of compost such as self-heating test, temperature decline, color and odor, C/N ratio and other chemical tests are significant. Among them temperature decline is taken into account in this study. The major advantages of composting are the production of a stabilized end product that can be stored or spread with little odor or insect breeding potential, effective hygienization of the pathogenic bacteria present in the organic waste and decomposition of organic fraction of wastes to reduce its volume, weight and moisture content. The application of compost to agricultural lands brings significant favourable changes in the soil properties such increase in organic content, moisture retention capacity, soil texture and fertility of the soil. Thus the study has a great importance in reducing the volume of wastes generation as well as stabilizing the biodegradable organic kitchen wastes.

2. METHODOLOGY

2.1 Collection and Proportion of Wastes

The vegetable wastes from kitchen, paper waste and sawdust were selected for the solid waste mixture. The vegetable waste and paper waste were collected respectively from the dining and boarders of Rokeya Hall, Khulna University of Engineering & Technology. Sawdust was collected from a local saw mill. The large pieces of vegetable wastes and waste paper were cut into small pieces of size 1 to 1.5 cm. The selected proportion of waste mixture was 75:10:15 (kitchen wastes: waste paper: saw dust). Sawdust was used as bulking agent in the mixture. All the wastes were mixed uniformly.

2.2 Type of Reactor and Aerator

Thermo-fluxes were used as reactor. The diameter, height and capacity of each reactor were 10cm, 27cm and IL respectively as shown in Figure 1.

Figure 1: Measurement of temperature inside reactors

Six aerators (Super Pump SP-780) were used for aeration during composting. The air pipes of diameter of 5mm and length of lm were used to connect the aerator with the reactors. One aerator can aerate four reactors connected with four air pipes. The air was passed daily at the rate of 500 ml/min through the waste mixture inside each reactor for 5 hours in day time. The aerator and air pipe are shown in Figure 1.

2.3 Temperature Measurement

The thermometers of temperature range of 120 °C were used to monitor the temperature generated in the waste mixture inside the reactors due to composting. The thermometers were inserted into reactor for monitoring the temperature. The temperature readings were taken regularly as shown in Figure 1.

2.4 Determination of Moisture Content and Volatile Solids

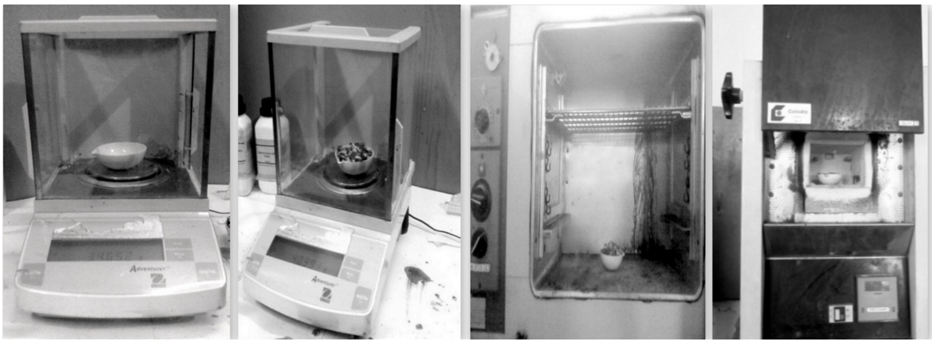

Figure 2: Determination of moisture content and volatile solids

Step by step determination of moisture content and volatile solids is shown in Figure 2.As can be seen in Figure 2, at first the weight of a small container (w_1) was measured using a digital balance. A small amount of waste sample was taken into container. The weight of the wet sample with container (w_2) was measured and then it was kept in the oven at 105 ± 5 °C during 24 hours. The weight of the oven dried sample with container (w_3) was measured. Then the moisture content was calculated by using following formula:

Moisture content, M.C. (%) $= \{(w_2-w_3) / (w_2-w_1)\} \times 100\%$

The oven dried sample was burnt in Muffle Furnace at 550 °C during 3 hours. Then the weight of the fixed sample with container (w_4) was measured. Then the volatile solids was calculated by using following formula:

Volatile solids, V.S. (%) $= \{(w_3-w_4) / (w_3-w_1)\} \times 100\%$

2.5 Procedure for First and Second Run

The first run was performed using three reactors with the passive aeration composting. The reactors were filled with the waste mixture and shacked gently. On the other hand, the second run was performed using three reactors with forced aeration composting and before filling the reactors with waste mixture, air pipes from air pump were connected inside each reactor. Then the reactors were filled with waste mixture and shacked gently. The air was passed at the rate of 500 ml/min through the waste mixture inside reactors for 5 hours in day time daily except the holidays. In both run the openings of the reactors were closed by small pieces of polyeurathane sheet and thermometers were inserted into them for monitoring the temperature readings. The total sample weight, moisture content and volatile solids of the waste mixture were determined before and after composting. The temperature readings were taken daily during 29 days for each run until the temperature fell near ambient temperature. Experimental set up during first and second run is shown in Figure 3 and 4.

Figure 3: Experimental setup during first run

Figure 4: Experimental setup during second run

2.6 Procedure for Third and Fourth Run

During the third run volatile solids degradation rate was determined using twenty (20) reactors by forced aeration composting process selected from first and second run. All the process was same as second run. The experimental set up is shown in Figure 5.

Figure 5: Experimental set up during third run

The temperature readings were taken daily for 24 days. The total sample weight, moisture content and volatile solids of the waste mixture were determined at 3, 4 days interval at first four weeks. After four weeks the total sample weight, moisture content and volatile solids of the waste mixture were determined by opening three reactors in each week. One was three days interval and other two reactors were four days from the first one of that week. This procedure was continued up to twenty numbers of reactors. The determination of volatile solids degradation rate was completed within 56 days. Then the fourth run was performed using seven (7) reactors to determine the maturity of compost. The fresh compost sample from third run was used as feed materials for the fourth run for determination of temperature variations. The experimental set up process was same as the second run. The temperature readings were taken daily for 16 days until the temperature fell near ambient temperature. The experimental set up is shown in Figure 6.

Figure 6: Experimental set up during fourth run

3. RESULTS AND DISCUSSION

All the results are tabulated. The variations of different data are also represented by graph. The temperature variations with time for passive aeration composting during Run-1 are shown in Figure 7.

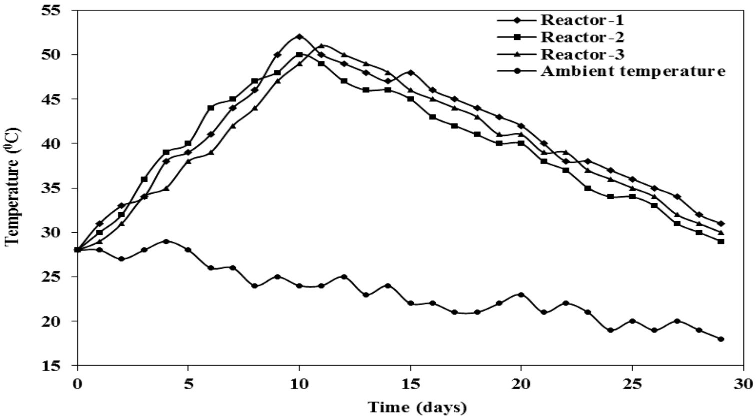

Figure 7: Variation of temperature with time during passive aeration composting for reactor-1, 2 and 3

Figure 7 illustrates that, initially the temperature inside reactors increased rapidly from 28 ^{0}C to 45 ^{0}C within 7 days and maximum temperature was 52 ^{0}C which was in the range of composting process (45 to 65 ^{0}C). Then the temperature decreased slowly up to 29 ^{0}C. Hence, it can be said that initially the temperature inside reactors increased rapidly for few days and then decreased slowly for the passive aeration composting. The temperature variations with time for forced aeration composting during Run-2 are shown in Figure 8.

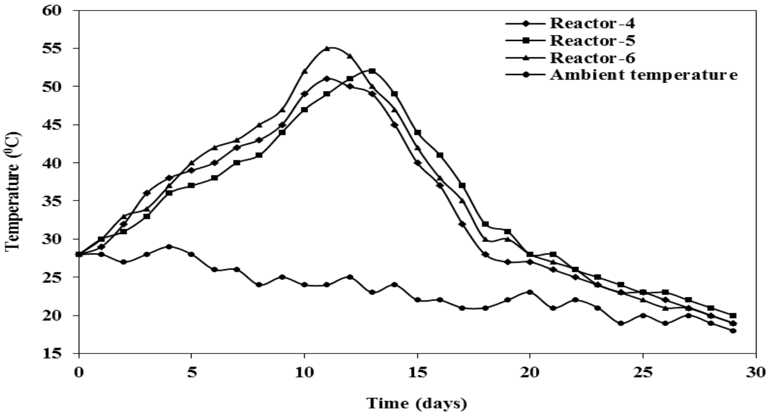

Figure 81: Variation of temperature with time during forced aeration composting for reactor- 4, 5 and 6

As can be seen in Figure 8, initially the temperature inside reactors increased rapidly from 28 ^{0}C to 43 ^{0}C within 7 days and maximum temperature was 55 ^{0}C which was in the range of composting process (45 to 65 ^{0}C). After that the temperature decreased rapidly for few days and then slowly up to 19 ^{0}C which was near ambient temperature. Hence it can be seen that initially the temperature inside reactors increased rapidly for few days and then decreased rapidly for few days and then slowly for the forced aeration composting. In both run, the maximum temperature was below 60 ^{0}C. That is why the organic waste mixture can be taken for the composting process to determine the volatile solids degradation with time. The changes in total sample, moisture content and volatile solids at initial and final conditions for the determination of temperature variation with time for the passive aeration composting during Run-1 and forced aeration composting during Run-2 are detailed in Table 1. As can be observed from the Table 1, during Run-1 in reactor-1, 2 and 3, the initial weights of the waste mixture were 390, 395 and 385 g respectively and after 29 days of composting the final weights of waste mixture were 315, 325 and 305 g respectively. Initially the moisture content of the waste mixture was 68.2% and finally increased up to 78.46%. Initially the volatile solid of the waste mixture was 93.67% and finally decreased up to 89.15%. The average percentage reductions in total sample, amount of water, and volatile solids were obtained 19.24%, 25.88% and 40.54% respectively. On the other hand during Run-2 in reactor-4, 5 and 6, the initial weights of the waste mixture were 365, 370 and 390 g respectively and after 29 days of composting the final weights of waste mixture were 185, 210 and 215 g respectively. Initially the moisture content of the waste mixture was 68.2% and finally decreased up to 65.9%. Initially the volatile solid of the waste mixture was 93.67 and finally decreased up to 87.52%. The average percentage reductions in total sample, amount of water, and volatile solids were obtained 45.81%, 48.1% and 44.32% respectively. As the average percentage reduction values of waste mixture were found to be greater for the forced aeration composting than that of passive aeration composting, the forced aeration composting of organic waste mixture was selected to determine as well as maturity of compost. However, the variations of temperature with time for the determination of volatile solids degradation rate by forced aeration composting during Run-3 are shown in Figure 11, 12, 13 and 14 for reactor-1 to 5, reactor-6 to 10, reactor-11 to 15 and reactor-16 to 20 respectively.

Table 1: Changes in total sample, moisture content and volatile solids for the determination of temperature variation with time for Run-1 and Run-2

Run		**1**			**2**		
Method		Passive aeration composting			Forced aeration composting		
Reactor		1	2	3	4	5	6
Moisture Content, %	Initial	68.2	68.2	68.2	68.2	68.2	68.2
	Final	78.46	75.31	74.88	64.89	65.14	65.9
Volatile solids, %	Initial	93.67	93.67	93.67	93.67	93.67	93.67
	Final	89.59	89.9	89.15	87.52	89.43	87.68
Total Sample, g	Initial	390	395	385	365	370	390
	Final	315	325	305	185	210	215
% Reduction		19.23	17.72	20.78	49.32	43.24	44.87
Average Reduction, %		19.24			45.81		
Amount of Water, g	Initial	265.98	269.39	262.57	248.93	252.34	265.98
	Final	247.15	244.76	228.38	120.05	136.79	141.69
% Reduction		7.08	9.14	13.02	51.77	45.79	46.73
Average Reduction, %		25.88			48.1		
Dry Solids, g	Initial	124.02	125.61	122.43	116.07	117.66	124.02
	Final	67.85	80.24	76.62	64.95	73.21	73.31
% Reduction		45.29	36.12	37.42	44.04	37.78	40.89
Volatile solids, g	Initial	116.17	117.66	114.68	108.72	110.2	116.17
	Final	66.79	72.14	68.31	56.84	65.47	64.28
% Reduction		42.51	38.69	40.43	47.72	40.59	44.66
Average Reduction, %		40.54			44.32		

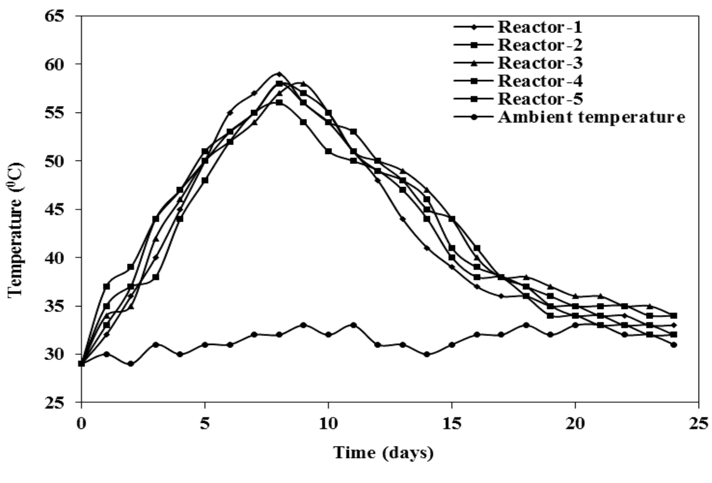

Figure 9: Variation of temperature with time during forced aeration composting for reactor- 1, 2, 3, 4, 5 and 6

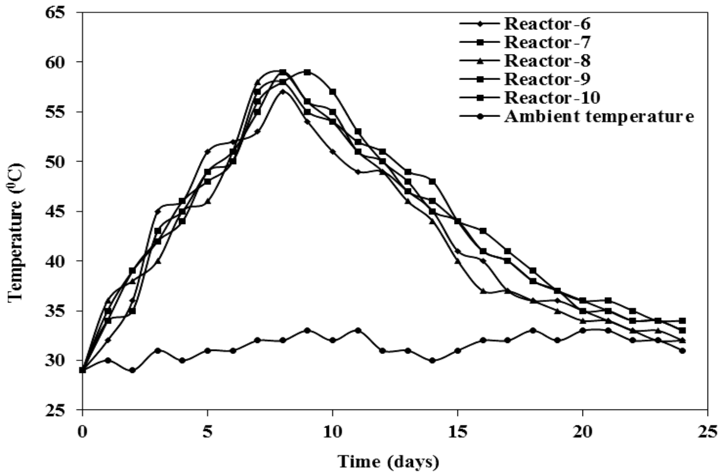

Figure 10: Variation of temperature with time during forced aeration composting for reactor- 6, 7, 8, 9 and 10

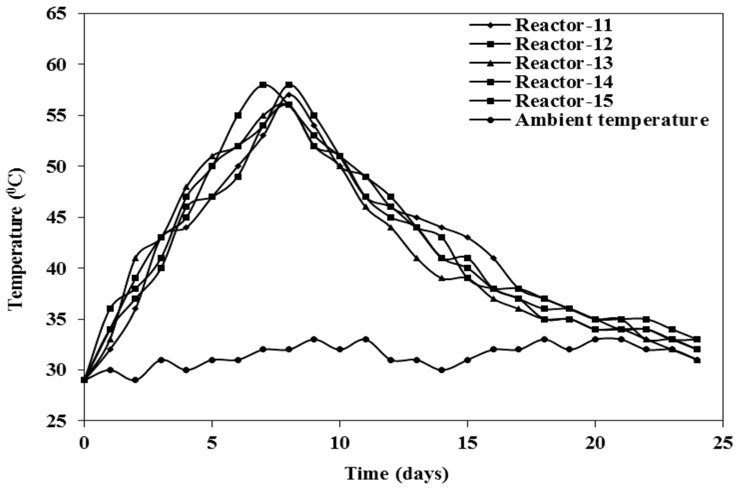

Figure 21: Variation of temperature with time during forced aeration composting for reactors 11to 15

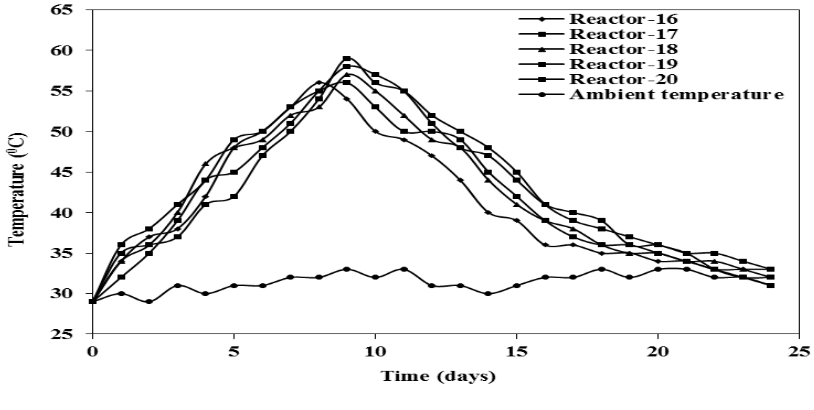

Figure 32: Variation of temperature with time during forced aeration composting for reactors 16 to 20

A cursory examination of the data plotted in Figures 9 to 12 show that the temperature increased from 29 ^{0}C to maximum 59 ^{0}C within 10 days and from average 7 to 11 days during composting the temperature was above 55 ^{0}C. Within these five days many of the microorganisms such as human or plant pathogens were destroyed because temperatures over about 65 ^{0}C kill many forms of disease causing microorganisms. Now the changes in total sample, moisture content and volatile solids for the determination of volatile solids degradation rate by forced aeration composting during Run-3 are presented in table 2, 3 and 4 for reactor-1 to 7, reactor-8 to14 and reactor-15 to 20 respectively.

Table 2: Changes in total sample, moisture content and volatile solids for the determination of volatile solids degradation rate for reactor-1 to 7 among 20 reactors during Run-3

Reactor		**1**	**2**	**3**	**4**	**5**	**6**	**7**
Time Interval, Days		4	3	4	3	4	3	4
Total Time, Days		4	7	11	14	18	21	25
Moisture Content, %	Initial	67.9	67.9	67.9	67.9	67.9	67.9	67.9
	Final	69.45	70.25	71.62	70.88	65.82	66.41	63.58
Volatile solids, %	Initial	93.4	93.4	93.4	93.4	93.4	93.4	93.4
	Final	92.18	91.86	91.33	90.95	91.51	91.16	90.91
Total Sample, g	Initial	290	350	370	410	350	330	315
	Final	280	325	335	360	275	240	210
% Reduction		3.44	7.14	9.46	12.19	21.43	27.27	33.33
Amount of Water, g	Initial	196.91	237.65	251.23	278.39	237.65	224.07	213.89
	Final	194.46	228.31	239.92	255.17	181.01	159.38	133.52
% Reduction		1.24	3.93	4.51	8.34	20.70	28.87	37.58
Dry Solids, g	Initial	93.09	112.35	118.77	131.61	112.35	105.93	101.11
	Final	85.54	96.69	95.08	104.83	93.99	80.62	76.48
% Reduction		8.11	13.94	19.95	20.35	16.34	23.89	24.36
Volatile solids, g	Initial	86.95	104.93	110.93	122.92	104.93	98.94	94.44
	Final	78.85	88.82	86.84	95.34	86.01	73.49	69.53
% Reduction		9.32	15.35	21.72	22.44	18.03	25.72	26.38

Table 3: Changes in total sample, moisture content and volatile solids for the determination of volatile solids degradation rate for reactor-8 to 14 among 20 reactors during Run-3

Reactor		**8**	**9**	**10**	**11**	**12**	**13**	**14**
Time Interval, Days		3	5	2	0	4	3	0
Total Time, Days		28	33	35	35	39	42	42
Moisture Content, %	Initial	67.9	67.9	67.9	67.9	67.9	67.9	67.9
	Final	62.12	60.95	57.41	56.73	59.04	58.96	56.72
Volatile solids, %	Initial	93.4	93.4	93.4	93.4	93.4	93.4	93.4
	Final	90.67	90.04	89.77	89.54	89.35	88.64	88.51
Total Sample, g	Initial	290	280	285	300	290	250	235
	Final	190	170	170	165	165	140	120
% Reduction		34.48	39.28	40.35	45.00	43.10	44.00	48.94
Amount of Water, g	Initial	196.91	190.12	193.52	203.70	196.91	169.75	159.57
	Final	118.03	103.62	97.59	93.61	97.42	82.54	68.06
% Reduction		40.06	45.50	49.57	54.05	50.53	51.38	57.35
Dry Solids, g	Initial	93.09	89.88	91.48	96.30	93.09	80.25	75.43
	Final	71.97	66.38	72.41	71.39	67.58	57.46	51.94
% Reduction		22.69	26.15	20.84	25.87	27.4	28.40	31.14
Volatile solids, g	Initial	86.95	83.95	85.44	89.94	86.95	74.95	70.45
	Final	65.26	59.77	65.08	63.92	60.38	50.93	45.97
% Reduction		24.95	28.81	23.83	28.93	30.56	32.05	34.75

Table 4: Changes in total sample, moisture content and volatile solids for the determination of volatile solids degradation rate for reactor-15 to 20 among 20 reactors during Run-3

Reactor		**15**	**16**	**17**	**18**	**19**	**20**
Time Interval, Days		5	2	0	3	4	0
Total Time, Days		47	49	49	52	56	56
Moisture Content, %	Initial	67.9	67.9	67.9	67.9	67.9	67.9
	Final	55.52	51.48	51.02	49.66	48.32	47.28
Volatile solids, %	Initial	93.4	93.4	93.4	93.4	93.4	93.4
	Final	88.13	87.92	88.85	88.72	88.47	88.35
Total Sample, g	Initial	250	225	230	280	255	225
	Final	135	85	80	120	110	85
% Reduction		46.00	62.22	65.21	57.14	56.86	62.22
Amount of Water, g	Initial	169.75	152.78	156.17	190.12	173.14	152.78
	Final	74.95	43.76	40.82	59.59	53.15	40.19
% Reduction		55.85	71.36	73.86	68.66	69.30	73.69
Dry Solids, g	Initial	80.25	72.22	73.83	89.88	81.86	72.22
	Final	60.05	41.24	39.18	60.41	56.85	44.81
% Reduction		25.17	42.89	46.93	32.78	30.55	37.95
Volatile solids, g	Initial	74.95	67.45	68.96	83.95	76.46	67.45
	Final	52.92	36.26	34.81	53.60	50.29	39.48
% Reduction		29.39	46.24	49.52	36.15	34.23	41.32

As presented in table 2, 3 and 4, from 56 days of composting during Run-3, initially the moisture content of the waste mixture was found 67.9% and finally increased up to 71.62% and decreased up to 47.28%. Initially the volatile solid of the waste mixture was obtained 93.4% and finally decreased up to 87.92%. The percentage reduction in total weight and moisture content were varied from 3.44% to 65.21% with an average value of 37.95% and 1.34% to 73.86% with an average value of 43.32% respectively. Hence the percentage reductions in total sample and moisture increased with time. The percentage reduction of volatile solids was varied from 9.32% to 49.52% with an average value of 28.98%. The volatile solids degradation rate is plotted in Figure 13 which illustrates that the percentage reduction in volatile solids increased with time. From the graph the rate of percentage reduction of volatile solids was found 0.52%. The value of coefficient of regression (R^2) was obtained 0.76 which is less than 1.So the percentage reduction of volatile solids is nearly accurate value.

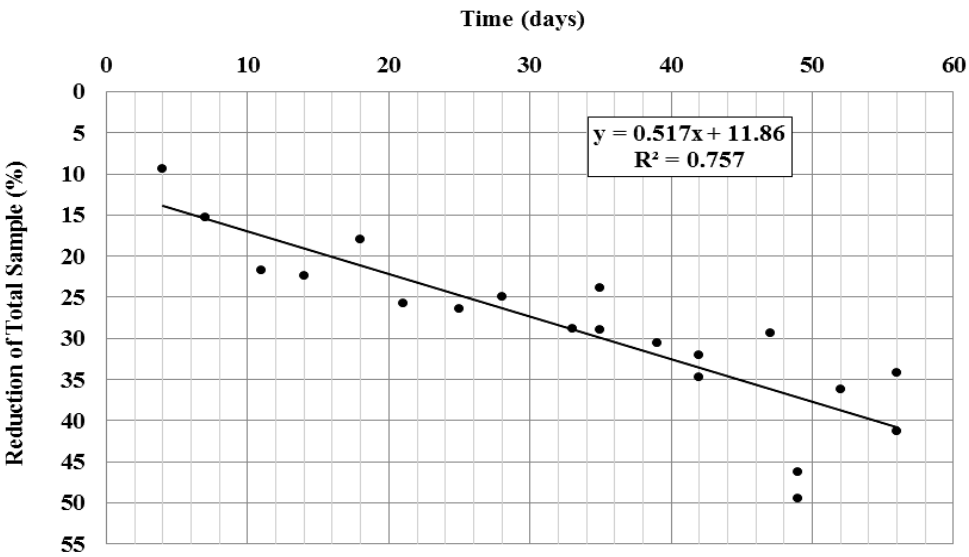

Figure 13: Volatile solids degradation rate during Run-3

However, the changes in total sample, moisture content and volatile solids for the determination of temperature variation with time during Run-4 are shown in table 5.

Table 5: Changes in total sample, moisture content and volatile solids for determination of temperature variation with time during Run-4

Reactor		**1**	**2**	**3**	**4**	**5**	**6**	**7**
Moisture Content, %	Initial	60.74	60.74	60.74	60.74	60.74	60.74	60.74
	Final	57.31	57.08	55.52	55.97	55.41	55.35	54.83
Volatile solids, %	Initial	88.26	88.26	88.26	88.26	88.26	88.26	88.26
	Final	87.34	87.23	86.41	87.02	87.29	86.17	85.44
Total Sample, g	Initial	280	270	290	275	220	285	210
	Final	250	235	245	235	185	245	175
% Reduction		10.71	12.96	15.52	14.54	15.91	14.04	16.67
Amount of Water, g	Initial	170.07	163.99	176.15	167.04	133.63	173.11	127.55
	Final	143.28	134.14	136.02	131.53	102.51	135.61	95.95
% Reduction		15.75	18.20	22.78	21.26	23.29	21.66	24.77
Dry Solids, g	Initial	109.93	106.01	113.85	107.96	86.37	111.89	82.45
	Final	106.72	100.86	108.98	103.47	82.49	109.39	79.05
% Reduction		2.92	4.86	4.27	4.16	4.49	2.23	4.12
Volatile solids, g	Initial	97.02	93.56	100.48	95.29	76.23	98.75	72.77
	Final	93.21	87.98	94.17	90.04	72.00	94.26	67.54
% Reduction		3.93	5.96	6.28	5.51	5.55	4.54	7.19

It is observed from the table 5 that initially the moisture content of the waste mixture was 60.74% and finally decreased up to 54.83%. Initially the volatile solid of the waste mixture was 88.26% and finally decreased up to 85.44%. The temperature variations with time for seven numbers of reactors during Run-4 are shown in Figure 14 which illustrates that the temperature inside reactors increased from 32 ^{0}C to maximum 48 ^{0}C within 7 days and after 16 days the temperature decreased slowly up to 34 ^{0}C. Initially the temperature inside reactors increased rapidly and then decreased slowly for the composting process.

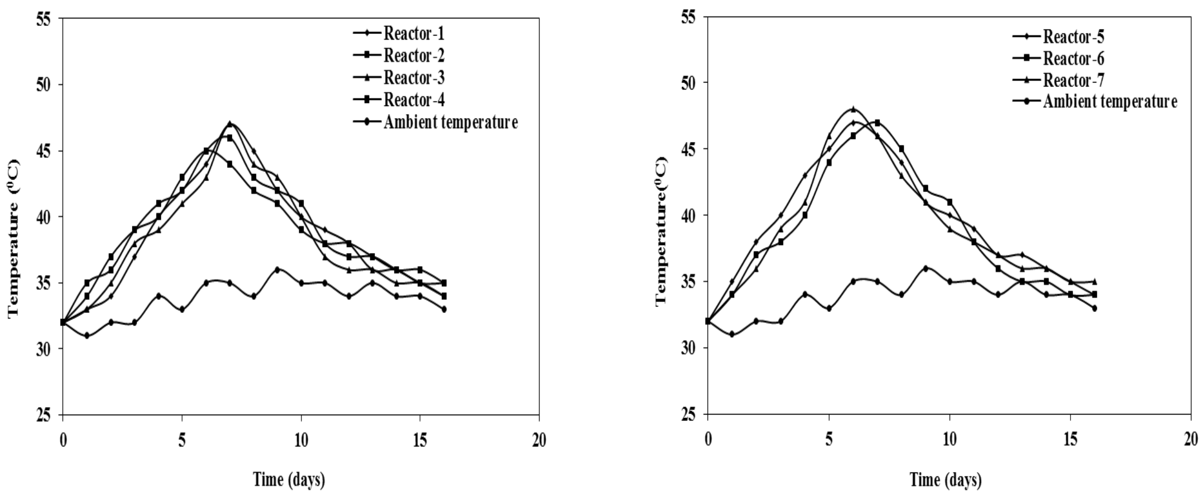

Figure 44: Variation of temperature with time during Run-4 for reactors1 to 7

Comparison between variation of temperature with time in Run-3 and Run-4 is shown in Figure 15. It is observed from Figure 15 that the basic difference between Run-3 and Run-4 was the temperature. During Run-3 the temperature was approximately 11 ^{0}C greater than that of Run-4 most of the time during composting period. So the energy in Run-3 was greater than in Run-4. Moreover, in case of Run-3 the temperature was fallen ±5 ^{0}C to the ambient temperature within 21 days whereas it took only 13 days in case of Run-4 to fall near ambient temperature.

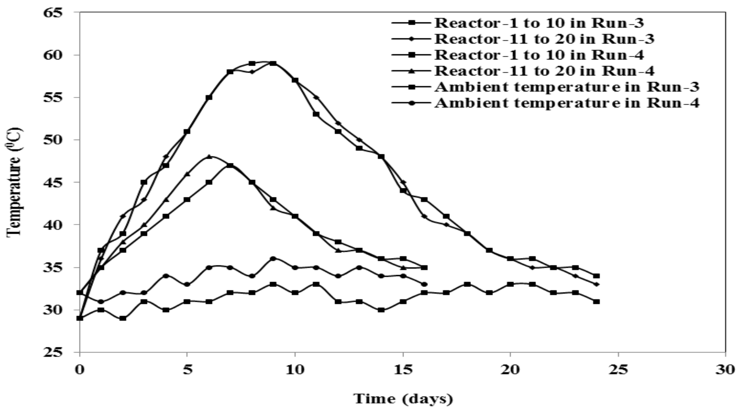

Figure 55: Comparisons of temperature variation between Run-3 and Run-4 for maximum temperature reading

4. CONCLUSIONS

This paper presents an analysis of effects of forced and passive aeration on composting process. From the analysis of the obtained data it was found that initially the temperature inside reactors increased rapidly for few days and then decreased slowly for the passive aeration composting during Run-1. On the other hand the temperature inside reactors increased rapidly for few days and then also decreased rapidly for few days and then slowly for the forced aeration composting during Run-2. In both run, the maximum temperature was above 55 ^{0}C. Again from the analysis of physical parameters of the waste mixtures, the average percentage reduction values of total sample, moisture content and volatile solids of waste mixture were found greater in forced aeration composting during Run-2 than in passive aeration composting during Run-1. That is why the forced aeration composting of organic waste mixture was selected to determine volatile solids degradation rate as well as maturity of compost. However, in case of determination of volatile solids degradation rate during Run-3, the percentage reduction in total weight, moisture content and volatile solids were varied from 3.44% to 65.21% with an average value of 37.95%, 1.34% to 73.86% with an average value of 43.32% and 9.32% to 49.52% with an average value of 28.98% respectively. From the cursory examination of the plotted graph, volatile solids degradation rate and the value of coefficient of regression were found 0.52% and 0.76 respectively. Again in this run, the temperature increased from 29 ^{0}C to maximum 59 ^{0}C within 10 days and from average 7 to 11 days during composting the temperature was above 55 ^{0}C within which days many of the disease causing microorganisms such as human or plant pathogens were destroyed whereas temperature inside reactors increased from 32 ^{0}C to maximum 48 ^{0}C within 7 days during Run-4. Hence it can be concluded that the temperature during Run-4 was smaller than in Run-3. So the compost found from fourth run was more mature than that of third run. Finally it was found that the percentage reduction in total weight, moisture content and volatile solids proceeded more with time which justifies composting of kitchen waste as a favourable solution for reduction of volume of solid wastes. However, it should be noted that the study is based on measurement of temperature variations and physical parameters of the waste mixtures. Further research is needed to investigate the effect of carbon-nitrogen ratio and oxygen requirement on composting process.

REFERENCES

Bari, Q. H. (2011) Forced Aeration Composting of Solid Waste: Theory Practice and Mathematical Models. LAP Lambert Academic Publishing. Germany.

Bari, Q. H. and Koenig, A. (2001) Effect of Air Recirculation and Reuse on Composting of Organic Solid Waste. *Resources Conservation & Recycling*, Vol. 33, pp. 93-111.

Epstein, E. (1997) *The Science of Composting*. Technomic Publishing Co. Inc., Lancaster.

Finstein, M. S., Miller, F. C. and Strom, P. F. (1986) Waste Treatment Composting as a Controlled System. In *Biotechnology*, VCH Verlagsgesellschaft GmbH, Weinheim, Germany, Vol 8, Microbial Degradations, pp. 363-398

Haug, R. T. (1993) *The Practical Handbook of Compost Engineering*. Lewis Publishers, Boca Raton.

Tchobanoglous, G. and Burton, F. L. (1995).Wastewater Engineering: Treatment, Disposal and Reuse. Metcalf & Eddy, Inc., Tata McGraw-Hill Publishing Company Limited, New Delhi.

Tchobanoglous, G.,Theisen, H. and Vigil, S. (1993). Integrated Solid Waste Management: Engineering Principles andManagement Issues. McGraw-Hill, Inc., New York.

DESIGN OF USER FRIENDLY ECO-SANITARY LATRINE

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ABSTRACT

Bangladesh has always to face pressing sanitation problems due to its vulnerable geographical location and lack of appropriate and adaptive technological options. Eco-sanitation toilets are found as one of the most appropriate and proven technological options, as these are cost effective, established and environmentally as well as socially sounds and reliable option, can effectively contribute in solving the existing and emerging sanitation problems of Bangladesh. However, the most common type of eco-latrine used in our country, double vault eco-latrine, has some problems with regards to ease of use and user friendliness. One major problem is the person using this type of latrine has to move from defecation pan to another pan for anal cleansing, which is very awkward and thought to be unhygienic in many areas of our country. These problems could be solved by modifying the current double pan system for defecation and anal cleansing. Only one pan has been designed for providing the facilities of defecation and anal cleansing without changing the position. Also the length of the footrest has been increased as the length of the pan so that the person using this latrine could easily move backward or forward for anal cleansing. Another modification has been done using mechanical lever and sliding commode for the same purpose. This modification does not change or hamper the current design of eco-latrine.

Keywords: Eco-latrine, Double vault eco-latrine, Two hole eco-pan, User friendly, Sliding commode

1. INTRODUCTION

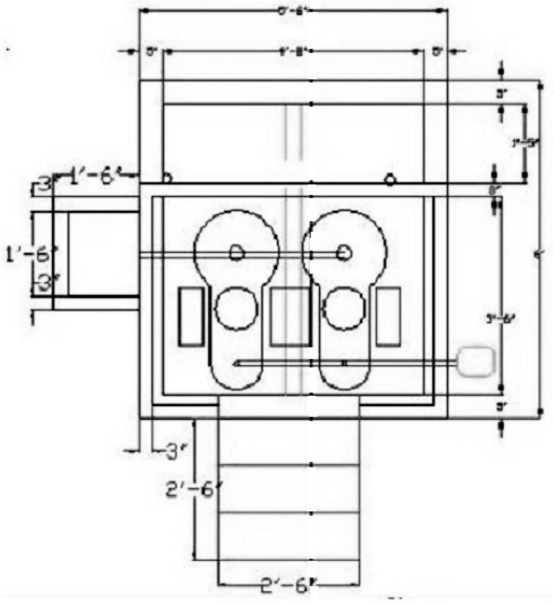
In the world, there are there are 1.1 billion people who lack access to a safe water supply, and 2.4 billion with no access to basic sanitation. In the next 25 years, the world's population is projected to grow by about another 2 billion people, most of who will be born in developing and emerging market economies and will be living in urban areas. Without a concerted effort, many of these people will be doomed to poverty. The limited progress in reducing poverty has many causes. Some of the most dramatic ones are directly related to our present situation of wastewater management and sanitation, which consists of using surface and groundwater as a sink for human excreta and wastewater, resulting in increasing health hazards, environmental and water pollution, the steady degradation of natural resources and also the permanent loss of nutrients and organics from the soil sphere (Werner et al., 2000). An alternative approach to avoid the disadvantages of conventional wastewater systems is eco- logical sanitation, 'eco-san' for short. This is based on an overall view of material flows as part of an ecologically and economically sustainable wastewater management systems tailored to local needs. It does not favor a specific technology, but constitutes a new philosophy in handling substances that have so far been seen merely as wastewater and water-carried waste for disposal (Werner et al., 2000).

Bangladesh has always to face pressing sanitation problems due to its vulnerable geographical location and lack of appropriate and adaptive technological options. Eco-sanitation toilets are found as one of the most appropriate and proven technological options, as these are cost effective, established and environmentally as well as socially sounds and reliable option, can effectively contribute in solving the existing and emerging sanitation problems of Bangladesh. Eco-san toilet has also been implemented in the flood prone areas. As the Eco-san toilets do not need water for cleaning and flashing, it also works fully during draughts. These entire situations clearly prove that the Eco-san toilets are adaptable with effects (Floods, draughts, tidal stage etc.) of climate changes (Biplob, et al., 2011). The general perceptions of people on ECOSAN were found encouraging. About 62% of the users felt excited whereas 23% expressed their satisfaction. However 12% expressed the need for further improvement. Similarly, the majority of the neighbors of the ECOSAN users (54%) expressed a positive perception of ECOSAN whereas 32% expressed satisfaction and 10% expressed the need for further improvement and 4% of them are not interested (Biplob, et al., 2011). The motivation for becoming attracted to the Eco-san toilet is mainly due to easy availability of fertilizers, permanent structure, sanitation and environmentally sound technology.

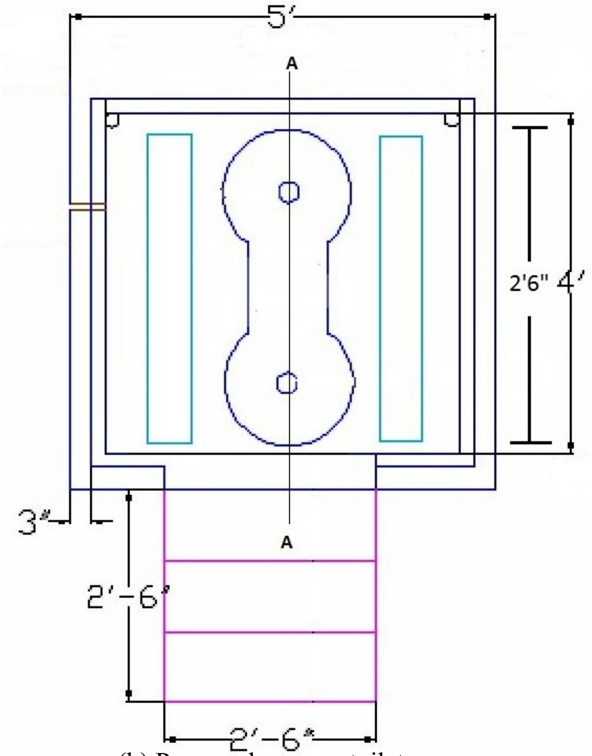
Eco-San toilet is a urine diversion toilet and based on the idea that urine, feces and water are resources in an ecological loop. It has two defecation holes at the top of each vault that receive feces. Sufficient ashes are used to cover new feces for protecting odor, flies, insects and pollutions. Moreover, it does not need to use water for cleaning and flashing. However, Eco-san can effectively contribute in safely transforming human urines and faces into high-potent organic fertilizers for eco-friendly agriculture and producing qualitative nutrient food-crops. The most common Eco-san technology, double-vault Eco-san toilets, prevent contamination of surface and ground water by processing feces in spaces that do not come into contact with water and prevent leakage through soil. But one major disadvantage is, the person who uses it has to move from one vault to another for anal cleansing after defecation, which in many parts of our country, especially in rural areas seems a bit odd or uncomfortable. The aim of this study is to solve this inconvenience with design modification of defecation pan and or with introducing some mechanical sliding arrangement.

2. METHODS

The modification of existing eco-latrine has been done in two methods. In the first method, two eco-pans have been assimilated in only one eco-pan to facilitate both defecation and urinating with anal cleansing. This one is a very cost effective and economical design as it does not require any extra mechanical arrangement. The footrest has been extended to the length of the eco-pan, so that the user can perform the act of anal cleansing by manually going forward or backward in the direction of the defecation and urinating with anal cleansing hole. The Figure 1(a) and 1(b) describe the existing conventional eco-san toilet and proposed eco-san toilet respectively with first method. The second method includes a mechanical lever and a commode over the sliding carriage to facilitate defecation and anal cleansing in one pan. In this method the user can move forward or backward by using a lever.



(a) Existing conventional eco-san toilet



(b) Proposed eco-san toilet

Figure 6: comparison of existing design to the modified design

2.1 Existing Design of Eco-Latrine

Existing conventional eco-latrine which is the most commonly used in our country is shown in Figure 2. Two plastic fiber eco-pans are used in this design, which could be alternatively used during the duration of 6 months. The total area required for this type of latrine is 33 square feet at maximum. The pans are situated within 6 inches along each other.

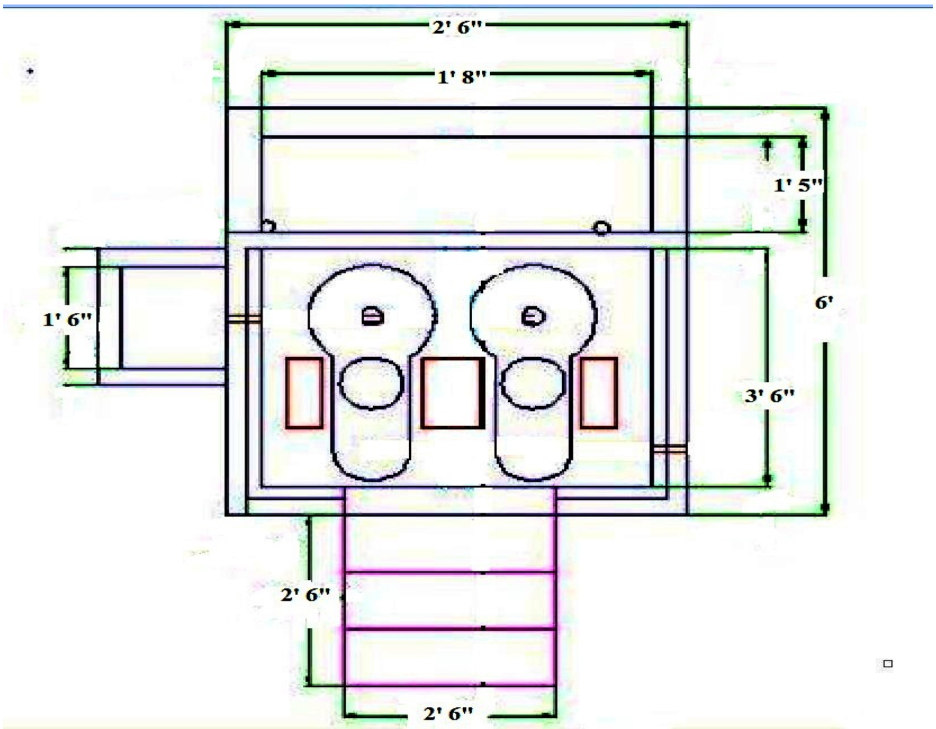


Figure 2: Existing design of eco-latrine

2.2 General Design Considerations of Eco-Latrine

The following conditions are considered for the design of an eco-latrine (DPHE and UNICEF, 2010):

- It could be constructed outside or inside the house.
- Avoid places that are submerged by flood water.
- The lower chamber or the substructure could be elevated to place the containers of excreta.
- The lower chamber should be properly enclosed to minimize infestation of insects and prevent the feces from getting wet.
- It should be provided with enough ventilation.
- It should be provided with access doors for easy retrieval of wastes and maintenance purposes
- Reinforced concrete slab should be preferred. Any other form of flooring materials is also acceptable.
- Any available and appropriate materials could be used for the walls, doors, windows and roof
- There should be a Urine-Diversion bowl and a provision for anal washing.
- It should be wide enough to contain the necessary fixtures and facilities.
- The space should be comfortable enough for the users.

2.3 Considerations for Design Modification

The following factors are considered for the design modification of existing eco-latrine:

- Two eco-pans are replaced by only one pan with two holes.
- The length of the footrest is made equal to the length of the pan.
- One chamber is used for feces storage.
- One chamber is used for water storage or separation.

2.4 Modified Design

The existing design of eco-latrine is modified for simplicity and ease of use. Figure 3 shows the cross section of the modified scheme. The maximum area required for this type of eco-latrine is 33 square feet. There are two chambers, one of them is used for feces storage and another is used for the storage of urine and anal cleansing water. Figure 4 shows the plan and Figure 5 shows the details three dimensional view of the modified eco-latrine. Two vent pipes from two corners are used to remove the odor from the toilet. The length of the pans is kept between 2 to 2.5 feet. The superstructure could be made between 7 feet to 10 feet height. One pan could be used for defecation and the other one for anal cleansing. The vaults beneath the pans have to be placed in way that they fit their purpose. Also the vault size beneath the anal cleansing pan has to be considerably bigger than the defecation vault.

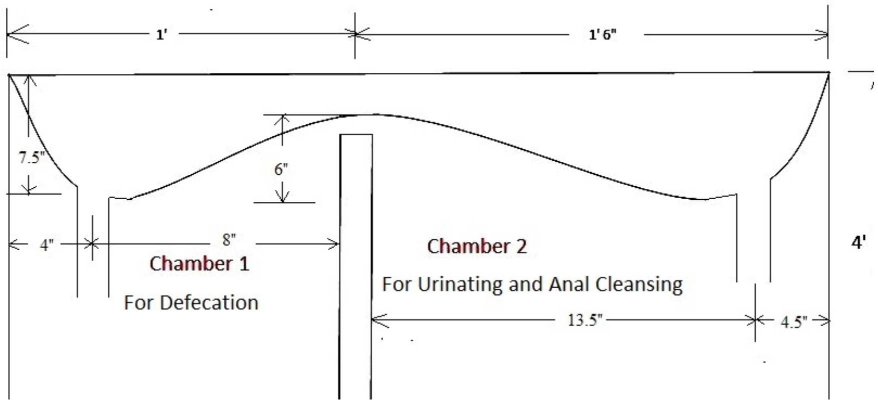
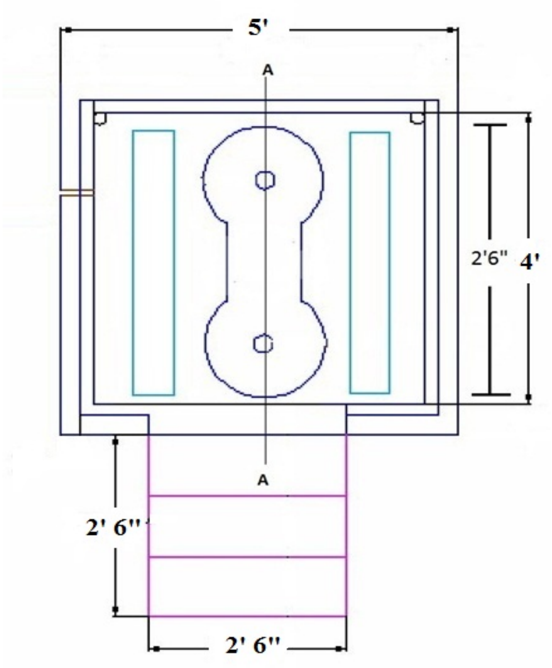


Figure 3: cross section of modified design



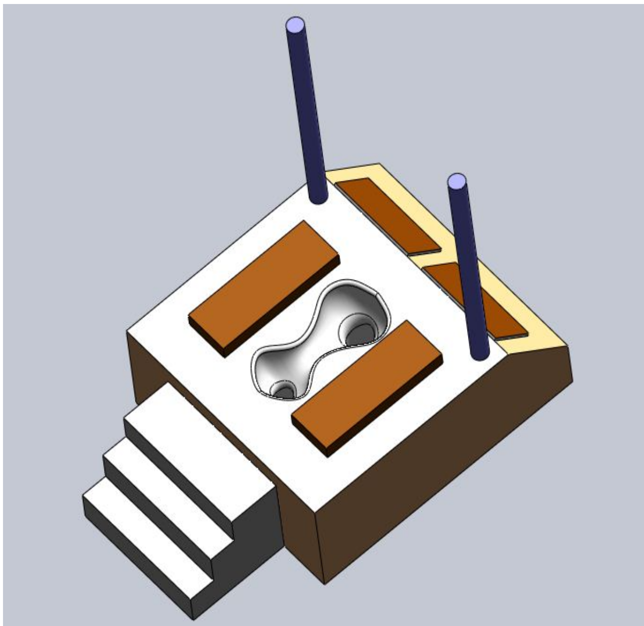


Figure 4: Details plan of modified design

Figure 5: Three dimensional illustration of design modification

2.5 Modification by using Movable or Sliding Commode

Using the facility of mechanical arrangement, the design could be modified. This regime includes using a hand powered or electric powered wheel which resides by the footrest and could be operated manually. The user has to revolve the wheel and the movable footrest would move along the pan. This system could be highly useful for elder persons. The movement of the sliding commode is facilitated by a hand driven roller. Fig 7 shows the action of hand driven lever. When any user circulates the lever, the sliding commode moves over the eco-pan with the lever. This gives a user the facility of defecation and anal cleansing in one single pan without doing any significant movement. This is a significant improvement over the previous modification.

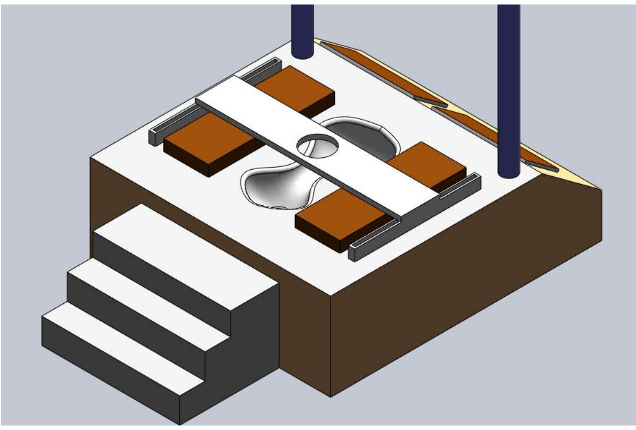


Figure 6: Modification using sliding commode

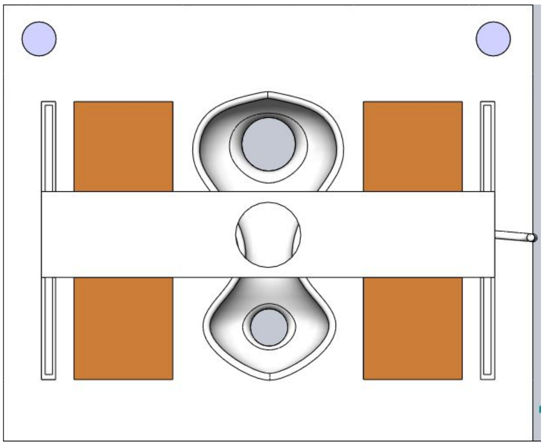


Figure 7: Action of a hand driven lever

The following table shows the dimensions of various components used in the design

Table 1 Details of design components

Component	Dimension
1. Superstructure	10 feet
2. Pan area	30 square feet
3. Length of pan	2.5 feet
4. Width of pan	1.5 feet
5. Depth of pan	7.5 inch
6. Total depth of chamber (with vault)	4 feet
7. No of vent pipes	2

3. CONCLUSIONS

The main modification was made by facilitating both anal cleansing and defecation in one single eco-pan. Another modification is made by using a mechanical lever for movement of the user throughout the defecation and anal cleansing process. The modified design eco-san toilet would achieve the advantages over the conventional eco-san toilet are ease of use, defecation and anal cleansing in a single pan and safer for elders and children.

REFERENCES

Biplob Pramanik, Ram Chandra Sarker, Dipok Chandra Sarker: Eco-san Toilet for Sustainable Sanitation Practice in Bangladesh. International Journal of Civil & Environmental Engineering IJCEE-IJENS Vol: 11 No: 05; Page no. 1-3

Werner Christine, GTZ Eco-san project, April 2003: 2'nd international symposium on ecological sanitation, Dag-Hammarskjöld-Weg 1-5, 65726 Eschborn, Germany; page no. 24-30.

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PLASTIC WASTE RECYCLING SCENARIO IN KHULNA CITY

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ABSTRACT

Plastics are inexpensive, lightweight and durable materials, which can readily be molded into a variety of products that find use in a wide range of applications. Significant quantities of plastics are left in the residual waste stream and are disposed of in landfill. Stoppage of using plastic materials sounds irrational and would impose negative impact on the economy. Plastic waste management is basically a welfare and development matter that demands active public participation and awareness for its success. It is able to lessen environment pollution and odor nuisances, and eventually contibutes to a sustainable resource management. The existing management process of plastic waste recycling in Khulna city is scrutinized in the current study and a novel management process is proposed to increase the capacity. The scrutiny is performed through a structured questionnaire and itemizing the waste collected from different plastic industries in Khulna city. The study finds most of the plastic recycling industries as unregistered to the government and a gender discriminaiton therein in terms of wages. Female workers are getting Tk. 300-800 per week in contrast to the male workers who get Tk. 1000-2500. Child labors are getting very little amount of wage despite their prime contribution on collecting the plastic scraps from source point. Establishing a testing and quality control laboratory for the recycled plastic products deems crucial for a healthy promotion of the fledgling industry.

Keywords: Plastic wastes, recycling and reuse, sustainability management scheme, wage discrimination, women and children laborers

1. INTRODUCTION

Solid waste disposal is now recognized worldwide as a critical issue that demands immediate attention. Environmentally unsafe techniques have been either banned or forced to comply with numerous standards of operation. In addition, the throw-away ethics of modern societies combined with the decreasing available space in urban areas have helped aggravate the problem. As a result, finding suitable place to dispose of waste is becoming a much more difficult and expensive endeavor (Esa *et al.*, 2011). The management of solid waste represents a major economic and environmental issue throughout the world (Demirbas, 2010). Higher recycling rates for valuable materials from waste streams could play a significant role substituting for virgin material production and saving fossil resources (Tonini and Astrup, 2012). As the population and use of resources are higher in urban areas, the rate of waste generation is also high. Urban residents generate two to three times more solid waste than their fellow rural citizens. The urban areas of Asia now spend about US$25 billion on solid waste management per year, with this figure increasing to at least US$50 billion in 2025 (World Bank, 1999). The total population of developing countries accounts for more than 70% of the world's population (JICA, 2005). Waste management in these countries is of grave concern from two points of view. Firstly, the process of urbanization and population concentration that is inextricably linked to waste management issues is progressing at a pace that is much faster than was ever experienced by today's industrialized countries (DESA, 2011). The issue of waste management in developing countries, therefore, has emerged as a critical and impending disaster. Secondly, these countries often have difficulty in streamlining the institutional systems, administrative bodies, management capabilities and human resources that are needed to take the lead in solving solid waste problems (UNEP, 2009). The composition of Municipal Solid Waste (MSW) in six major cities of Bangladesh is as follows: the food and vegetables waste ranges from 68 to 81%, while paper and plastic are 7.2 to 10.7% and 2.8 to 4.3% respectively (Waste safe, 2005). The remaining portions are rubber, cloth, metal, tin glass, dust and others. The fermentable portion is normally very high as compared to other portions in Bangladesh. In Bangladesh, India the putrescible portion is 7.5% and paper, metal, glass, plastics/rubber/leather, textiles and ceramics/dust/stone are 1.5%, 0.1%, 0.2% 0.9%, 3.1% and 19% respectively and in Australia the putrescible portion is 23.6% and paper, metal, glass, plastics/rubber/leather and ceramics /dust/stone are 39.1%, 6.6%, 10.2%, 9.9% and 9% respectively (Diaz *et al.,* 1996). The principles difference between waste generated in developing nations and those generated in industrialized countries is the higher organic content characteristic in developing nations. Now from the composition of solid waste which portion is recyclable is important. Since the 1970s, the consumption of plastics has grown dramatically and, consequently, so has the creation of waste plastics. Associated with this growth and reflecting changes in production and consumption, the composition of the waste bin has also changed; the proportion of organic matter has declined, while plastics have increased (ACRR, 2004). Plastics have substantial benefits in terms of their low weight, durability and lower cost relative to many other material types (Andrady & Neal 2009; Thompson *et al*., 2009a). Worldwide polymer production was estimated to be 260 million metric tonnes per annum in the year 2007 for all polymers including thermoplastics, thermoset plastics, adhesives and coatings, but not synthetic fibres (PlasticsEurope, 2008). This indicates a historical growth rate of about 9 per cent p.a. Thermoplastic resins constitute around two-thirds of this production and their usage is growing at about 5 per cent p.a. globally (Andrady, 2003). Today, plastics are almost completely derived from petrochemicals produced from fossil oil and gas. Around 4 per cent of annual petroleum production is converted directly into plastics from petrochemical feedstock (British Plastics Federation 2008). As the manufacture of plastics also requires energy, its production is responsible for the consumption of a similar additional quantity of fossil fuels. However, it can also be argued that use of lightweight plastics can reduce usage of fossil fuels, for example in transport applications when plastics replace heavier conventional materials such as steel (Andrady & Neal, 2009;Thompson *et al.,* 2009b). Khulna, the third largest city of Bangladesh with a large population (1.50 million) has been a place of commercial importance for more than 150 years. Management of solid waste in the municipal area is the responsibility of KCC. Average total per capita waste generation rate of KCC area is estimated at 0.22 kg/cap/day (Aborjona O Paribesh, 2000). Total waste generation is calculated at 200 tons/day where plastic waste forms almost 40% of this total waste (PREGA, 2005). This Large quantity of plastic contents present in the Khulna's waste composition indicates the necessity for frequent collection and removal. This also indicates the potentials of recycling of plastic waste for resource recovery. The main motive of this study is to introduce a sustainable management process for plastic waste recycling which is beneficial in economic consideration and defend the working environment from its harmful effect.

2. METHODOLOGY

2.1 Selection of study area

Khulna, the third largest city of Bangladesh, is located in the southern part of the country and is situated below the tropic of cancer, around the intersection of latitude 22.49^0N and longitude 89.34^0E. The area of Khulna city is 47 square km with a population 1.5 million (BBS, 2009). With regards to investigating the activities of plastic waste recycling, a field survey was conducted in the Khulna city area. Most of the recycling industries are located in Dowlatpur, Shekhpara, Sonadanga, Gollamari, Goalkhali commercial areas. The survey was conducted in Sonadanga, Gollamari and Goalkhali areas.

2.2 Data Collection & Analysis

Both primary and secondary data were collected in doing this research. Primary data, such as the opinion from waste collectors, recyclable waste dealers, industry workers and KCC officials through in depth interview. Secondary data, such as statistics and reports on the quantity of solid waste generated and its composition and management practices of Khulna, was collected from past study reports, books and journals etc. The data has been interpreted in a quite simple and straightforward way. The important points were noted, sorted and classified from the information obtained from observations and interviews.

3. RESULTS AND DISCUSSION

3.1 Plastic waste recycling industries in Khulna city

Plastic recycling industry is one of the new and multi-usable industries in Bangladesh. Because of its nature of work and contribution towards pollution reduction, it is called the green industry. Developed countries have established it many years before knowing its usability and necessity of keeping environment pollution free, dirt free and make country beautiful. In Bangladesh, the recycling industry is still developing and its quality is not up to the mark yet. In Khulna city the condition is even worse. There are about 11 plastic recycling industries in Khulna. Most of them are still unregistered to the government. Due to this, those industries are out of coverage of the government regulation, facilities and programmes. However, they have become a earning place for the people of low economic profile for their need of low initial capital. These industries can adopt an unlimited source of laboreres in return of low wage of labor. The recyclers collect raw-materials from retailers at a low cost and after recycling they send them to the manufacturing industries (Table 1).

Table 1: Daily Quantities of Recyclable Plastics Collected in Khulna city

Collectors	Quantity Collected per day (kg)	Percentage of Recyclable Materials
Wastebin-street children	150-240	9.09-14.54
Street-hawkers	450-600	7.05-9.39
Makeshift-vans	1500-2000	12.19-16.26
Wholesale shops	960-1500	8-12

Daily generation of MSW is 465 tons in Khulna city (Bari *et al.*, 2009). Plastic is the major (62.87%) generated inorganic waste among them. Other generated inorganic wastes are Iron, Aluminum and Tin (24.69%), Glass (11.23%) and Battery (1.21%). The estimated generation of different types of inorganic waste in Khulna city is shown in Figure 1.

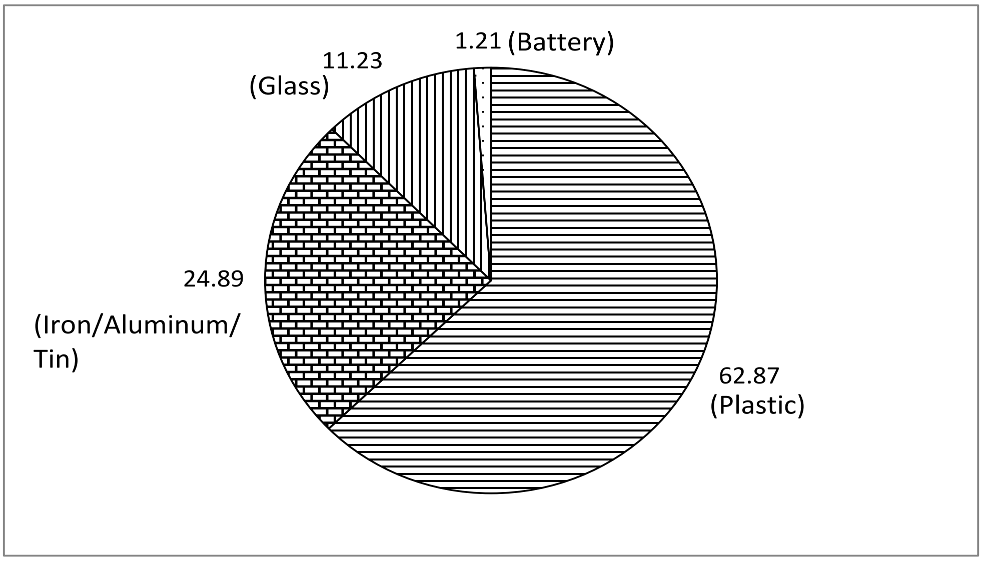

Figure 7: Composition of inorganic waste generated in Khulna (Afrin *et al.*, 2009 & Bari *et al.*, 2009)

The plastic wastes can be collected for recycling from people in residential areas by putting recycling plastic waste bins in vantage places for easy collection later and also collecting from the roadside. With the industrial plastic wastes, these can be collected from the industry the defects the plastic products and wastes. The processes of plastic recycling in steps are shown in Figure 2-6. Recycling has been practiced in the whole city region of Khulna. The areas mainly Daulatpur, Khalishpur, Shekhpara, Sonadanga, Gollamari were surveyed. Khalishpur and Shekhparaare industrial areas of Khulna. Almost all of the recycling industries are found in these areas. Dowlatpur, Shekhpara, Sonadanga, Gollamari are commercial areas. Comparison of plastic recyclable solid waste in different plastic industries has been shown in Table 2.

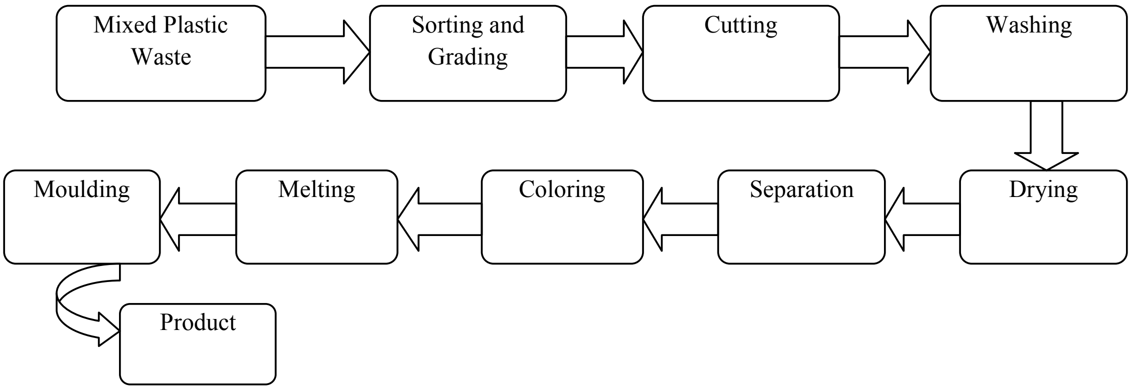

Figure 2: Typical flow diagram for the plastic recycling industry

Figure 3: Selected plastic waste

Figure 4: Plastic waste names COTTCA

Figure 5: Recyclable solid waste after machined

Figure 6: Prepared recyclable solid waste for supply to the buyers

All of the ten different items of plastic waste (viz. Domestic plastics, toy plastics, milk bottles, gas plastics, mineral water bottles, toilet cleaning containers, juice packs, hospital plastics, pen, and toothpaste tubes) segregated during the study. Domestic plastics (e.g. plastic pot, plastic plate etc.) was found the highest (28%) in Mukta plastic industries followed by 26% for Anika plastic industries. Toy plastics was found highest 24% in both Anika plastic industries and Tamim plastic industries followed by 21% in Metro plastic industries. Gas plastics (such as plastic gas cans) was found maximum (7%) in both Metro plastic industries and Tamim plastic industries. Mineral water bottle was found maximum (10%) in Metro plastic industries. Toilet cleaning containers wasfound maximum (9%) in both Anika plastic industries and Mukta plastic industries. Juice packs, hospital plastics, pen, toothpaste tubes was found maximum12%, 4%, 4% and 6% in Metro Plastic Industries, Tamim Plastic Industries, respectively. Table 2 also reveals that the value of toothpaste tube (tk/kg) was more than other waste.

Table 2: Comparison of Plastic Recyclable Solid Waste

Note: pp=plastic, wt=weight

Waste Types	Anika Plastic Industries		Metro Plastic Industries		Mukta Plastic Industries		Tamim Plastic Industries	
	% by wt	Value Tk/kg	% by wt	Value Tk/kg	% by wt	Value Tk/kg	% by wt	Value Tk/kg
Domestic pp	26	35	23	35	28	36	25	35
Toy pp	24	37	21	36	20	34	24	35
Milk bottle	8	37	6	44	10	34	9	37
Gas pp	6	55	7	61	5	58	7	55
Mineral water bottle	9	55	10	54	8	56	8	53
Toilet cleaning containers	9	52	08	58	9	53	8	52
Juice packs	7	57	12	64	8	56	5	57
Hospital pp	3	40	4	42	4	40	4	38
Pen	3	44	2	45	2	44	4	45
Toothpaste tubes	5	62	4	58	6	60	6	60

3.2 Recycling stages

Plastic Recycling Trade Chain includes different stakeholders at each stage. Huge numbers of laborer are engaged in these stages for their earning sources. These stakeholders include street-hawkers, Van-Collectors, Wastebin-street-children, DCC-Collectors, Dumpsite-street-children, makeshift-vans, Wholesalers and Manufacturers etc. The cycle in Figure 7 shows that how the plastic recycled from household to market. In this cycling process there create huge number of earning sources for different stakeholders. But from the field study, it is found that between household and collectors; there involve huge number of child-laborers for their earnings. On the other hand, women workers are mostly involved in recycling factories. In some cases, they are also involved in the scrap collection process.

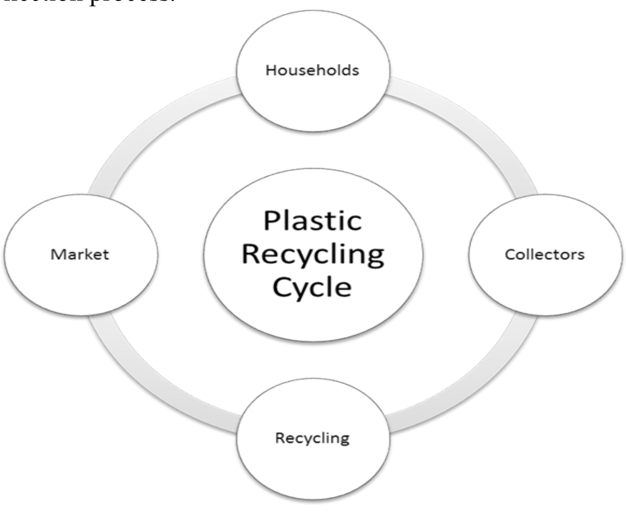

Figure 7: Plastic Recycling Cycle

3.3 Wage discrimination in the recycling industry

From the field survey it is found that there have intensive wage discriminations in plastic recycling sector. In scrap collection, there engage both male and female worker. Though they have to do same hard works, but they have severe discrimination in their wages. Huge numbers of child-laborers are also engaged in the door-to-door plastic scrap collection. Their wage scenario is even worse. They are getting very little amount of wage though they are the main source of collection of plastic scraps. They get wage only Tk.50-150 for their whole week hard work. But a male and female worker get wage respectively Tk.1000-2500 and Tk.300-800. So, the child laborers are one kind of exploited. Figure 8 shows a significant ratio gap between child-labor and labor wages.

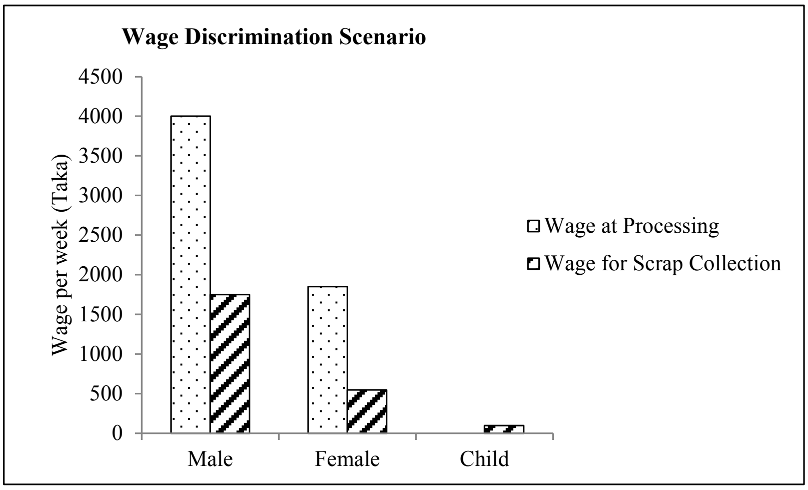

Figure 8: Wage Discrimination between male, female and child labor in plastic recycling sector

3.4 Different stakeholders at plastic-recycling process

Figure 9 shows the various stages of plastic recycling process and also the employment opportunity for different stakeholders at each stage. From the field study, it is found that the women-labors are mainly involved at the local recycling factories for grading, washing and grinding processes.

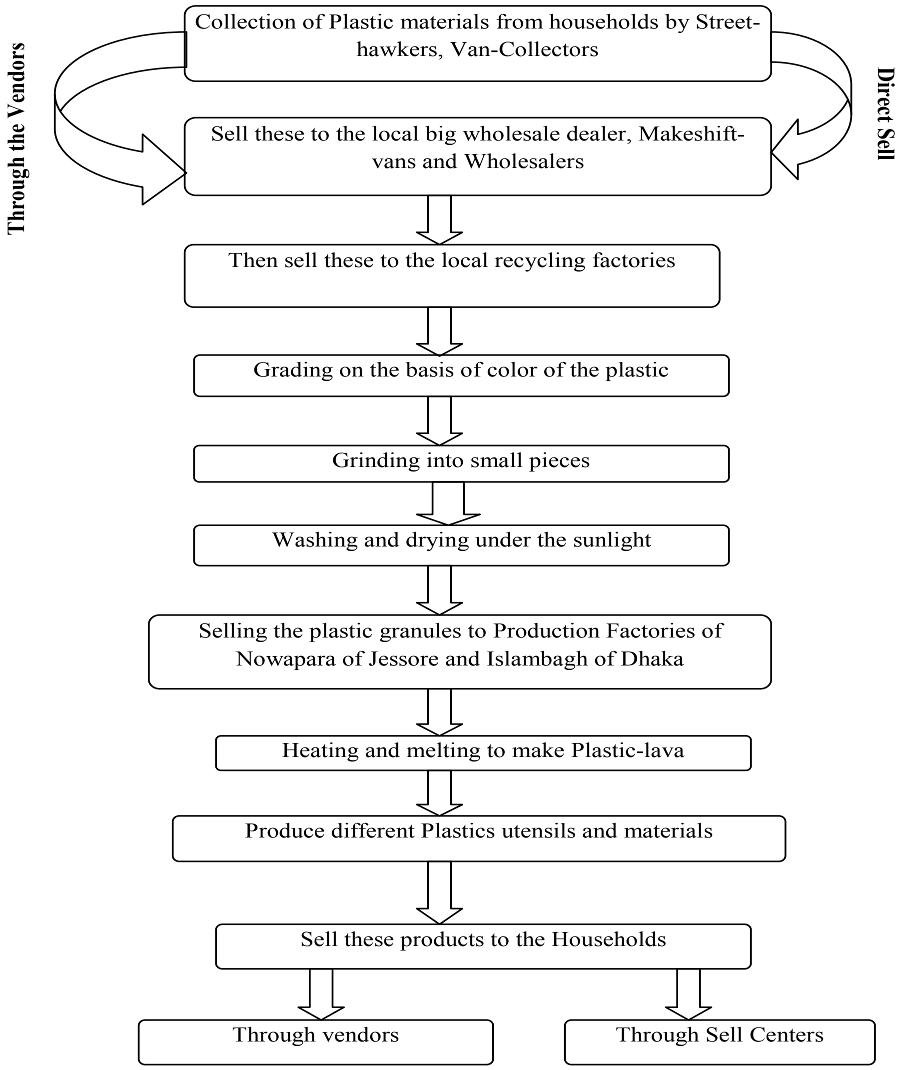

Figure 8: Plastic recycling stages

4. PROPOSED MODEL

Recycling is clearly a waste-management strategy. In a natural ecosystem there should be no wastes but only products (Frosch & Gallopoulos, 1989; McDonough & Braungart, 2002). The potentials of recycling of plastic waste have made it a growing business worldwide, both from economic and environmental point of view. In Bangladesh, plastic waste recycling is still based on rudimentary technology, mostly by informal sector. A study in 2005 shows that 45 percent of the plastic waste is recycled in the country by informal sector which resulted a savings of US $350 million in the year 2005 by avoiding export of virgin resin (Country Analysis Paper-Bangladesh, 2011). In order to improve the plastic recycling industry, the existing recycling process needs to be upgraded by eliminating its shortcomings. The workflow of a new model is shown in Figure 10.

In this process, each recycling industry should have at least one divisional unit at every district. The non-governmental organizations (NGO) or recycling factory agent of the Pouroshova or City Corporation in urban area would collect plastic waste from household/office. In rural area, thana collectors collect the waste and transfer these to pouroshova/city corporation. District branch collectors collect the waste and separate it according to types such as parts of bottles, container, toys etc. The parts of different plastics then send to the divisional recycling factory. After proper recycling process, the recycled parts send to the manufacturers. Manufacture company produce plastic products by different process such as injection molding (produce containers), blow molding (produce bottles) etc. New products go to the consumer hand and reuse it.

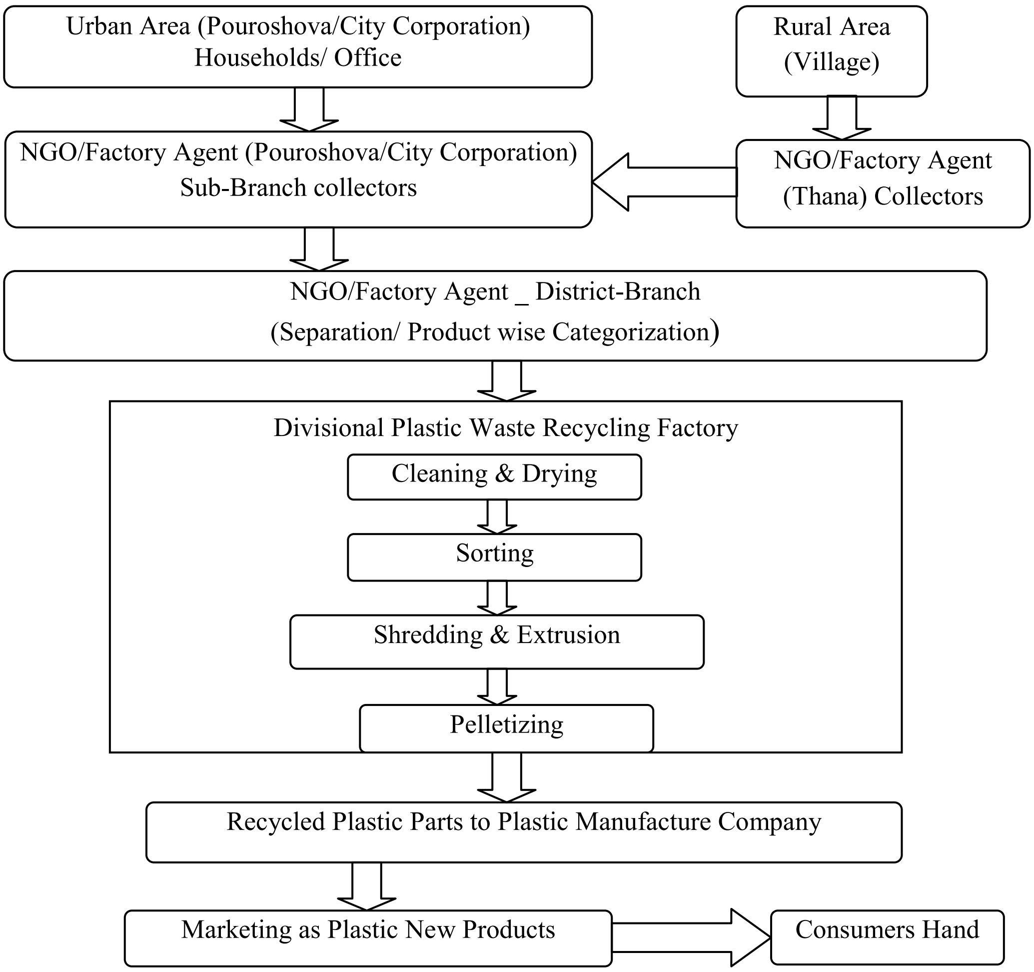

Figure 10: Proposed model for sustainable recycling of plastic wastes

5. CONCLUSIONS

The existing plastic recycling practice in Khulna city is studied in the current paper. This paper has provided some qualitative and quantitative information on plastic waste recycling sector in Khulna city. This study tried only to unfold a theoretical model for better plastic waste recycling process in Khulna city. The acclaimed management process is suitable for both urban and rural areas because, generated plastic waste is collected by NGO who are the affiliates of existing social system. They can go from door to door and can attract the people about the proposed model. This model is selected for divisional recycling and so burden is released from the capital city. The participation of NGOs can improved the overall MSW management system, especially waste collection process from sources and able to motivate the residents to store the waste properly and to keep clean the premises. In the existing process several plastic parts are decomposed without processing. This study tried to find a reliable system for proper recycling and management of produced plastic waste for further use. To investigate the possibility of this model, a complete empirical study is necessary.

REFERENCES

Aborjona O Paribesh, (Issue#5) (2000). A newsletter on waste management and recycling in Bangladesh.

Afrin, N., Rafizul, I. M. and Alamgir, M. (2009). "Commercial Opportunity of Composting, Recycling and Landfill of Municipal Solid Waste: Bangladesh Perspectives." Waste Safe2009, Vol. 2, 607-616.

Andrady, A. (2003). "An environmental primer. in Plastics and the environment.". Hoboken, NJ: Wiley Interscience, 3-76.

Andrady, A. L. & Neal, M. A. (2009). "Applications and societal benefits of plastics." Phil. Trans. R. Soc., Vol. B364, 1977– 1984.

Association of Cities and Regions for Recycling (2004). "Good Practices Guide on Waste Plastics Recycling a Guide by and for Local and Regional Authorities."

Bari, Q. H., Moniruzzaman, S. M., and Uddin, M.A. (2009). "Recycling of Solid Waste in Khulna City of Bangladesh." Waste Safe2009, Vol.2, 623-630.

BBS (2009). Statistical Pocket Book of Bangladesh. Bangladesh Bureau of Statistics. 8.

Demirbas, A. (2010). "Waste management, waste resource facilities and waste conversion processes." Energy Conversion and Management, Vol. 52, 1280–1287.

DESA (2011). "World Urbanization Prospects." Department of Economic and Social Affairs/Population Division, United Nations, New York.

Diaz, L. F., Savage, G. M., Eggerth, L. L., & Golueke, C. G. (1996). "Solid waste management for economically developing countries." Hercules, CA: Cal Recovery Inc.

Esa, A.J., Sharif, M.S., Faruque, M.A., (2011). "Analysis of the Recycling of Plastic Waste: A Case Study of Islambagh Area." Wastesafe-2011, Vol. 76,1-10.

Frosch, R. & Gallopoulos, N. (1989). "Strategies for manufacturing." Sci. Am. Vol. 261, 144–152.

JICA (2005). "Supporting Capacity Development in Solid Waste Management in Developing Countries – Towards Improving Solid Waste Management Capacity of Entire Societies." Japan International Cooperation Agency.

McDonough, W. & Braungart, M. (2002). "Cradle to cradle: remaking the way we make things New York." NY: North Point Press.

PlasticsEurope (2008). "The compelling facts about Plastics 2007: an analysis of plastics production, demand and recovery for 2007 in Europe." Brussels, Belgium.

PREGA (Promotion of Renewable Energy, Energy Efficiency and Greenhouse Gas Abatement) (2005). "Quantity and Quality Assessment of Khulna City Solid Waste for Electricity Generation-A Technical Study Report." PREGA National Technical Experts from Bangladesh Centre for Advanced Studies, Dhaka, Bangladesh.

Third Meeting of the Regional 3R Forum in Asia (2011). Country Analysis Paper (Draft)-Bangladesh. 6.

Thompson, R. C., Swan, S. H., Moore, C. J. & vom Saal, F. S. (2009a). "Our plastic age." Phil. Trans. R. Soc. Vol. B364, 1973–1976.

Thompson, R. C., Moore, C. J., vom Saal, F. S. & Swan, S. H., (2009b). "Plastics, the environment and human health: current consensus and future trends." Phil. Trans. R. Soc., Vol. B364, 2153–2166.

Tonini, D., Astrup, T. (2012). "Life-cycle assessment of a waste refinery process for enzymatic treatment of municipal solid waste." Waste Management, Vol. 32, 165–176.

UNEP (2009). "Developing Integrated Solid Waste Management Plan, Training Manual." Vol. 4. ISWM Plan, United Nations Environmental Programme, Osaka/ Shiga, Japan.

Waste Safe (2005). "Integrated Management and Safe Disposal of Municipal Solid Waste in Least Developed Asian Countries – A Feasibility Study." Department of Civil Engineering, KUET, Bangladesh & Asia Pro Eco Program of the European Commission.

World Bank, (1999). "What a waste: solid waste management in Asia, urban development sector unit. East Asia and Pacific Region." The International Bank for Reconstruction and Development, Washington DC, USA.

ENVIRONMENTAL COMPARISON BETWEEN STONE AND BRICK CHIPS CONSTRUCTION.

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ABSTRACT

This paper will show the environmental impact of a construction building and change of material will change the consumption of energy and carbon dioxide (CO_2) emission .Here mainly the emission of carbon dioxide (CO_2) is taken into consideration. The impact of global warming due to climate change is adverse for a river surrounded country like Bangladesh. We compare the use of stone and brick chips construction and find out overall impact of that material in construction. This paper categorically estimates the carbon dioxide (CO_2) emission from typically building plans for its construction life which hopefully helps us in guiding the reduction of carbon dioxide emission from a building. This research also focuses on the comparison of carbon dioxide (CO_2) emission from different construction materials of same quantity and comes out with a conclusion that use of bricks produces 3 times more emission than stones. This necessitates the using of alternate materials to achieve environment friendly green building with lesser emission of greenhouse gas like carbon dioxide (CO_2).

Keywords: Global warming, carbon dioxide (CO2), energy consumption, cost analysis, brick chips, stone chips.

1. INTRODUCTION

Concrete is the most largely consumed construction material worldwide and almost seventy percent of concrete volume is aggregate Brick chips are man-made coarse aggregate and Stone, especially granite, is extensively used in building construction as coarse aggregate . The manufacture of bricks is an energy-intensive activity. All developed countries and some developing ones have shifted away from traditional low-efficiency manufacturing processes to modem high-efficiency ones.

In Bangladesh, bricks are the predominant building material in urban areas. They have also become a significant building material in the rural areas. High prices or scarcity of alternative building materials, such as stones, iron sheets, wood, bamboo, and straw are very rapidly increasing the demand for bricks. In Bangladesh**,** about 8000 brick kilns are in operation, producing about 17.5billion bricks per year. Crushed stone aggregate is widely used for concrete manufacture. Considerable amount of fine granite dust is generated at the site of stone extraction whether it is manual shaping of stone or mechanized crushing.

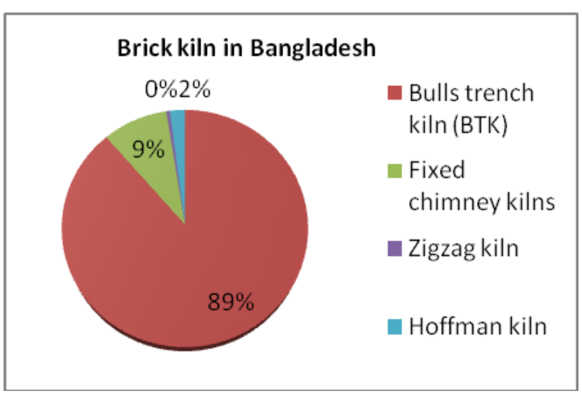


Figure1: Percentage of Brick Kiln in Bangladesh

2. OBJECTIVES

Objective of the thesis work is to estimate the amount of carbon dioxide (CO_2) emission for 6 storeys's building in Dhaka city for its entire life cycle in construction phase. Here two 6 storey building in Dhaka has analyzed.

- Estimation of unit carbon dioxide (CO_2) emission and energy consumption for different types of building materials.
- Estimation of quantity of building materials required in 6 storey building, thereby calculating carbon dioxide (CO_2) emission and energy consumption.
- Evaluate critical component of carbon dioxide (CO_2) emission for a building and suggest ways to reduce it.
- Comparison carbon dioxide (CO_2) and environmental impact between the building materials like stone & brick chips.
- Evaluate cost efficiency of brick and stone chips.

3. AGGREGATE

Concrete aggregate is a material which is mixed with cement to create concrete which is hard, strong, and long-lasting. The importance of using the right type and quality of aggregates cannot be overemphasized. The fine and coarse aggregates generally occupy 60% to 75% of the concrete volume (70% to 85% by mass) and strongly influence the concrete's freshly mixed and hardened properties, mixture proportions, and economy. But there are some basic differences in using different type of aggregate.

3.1.1 Comparison between Stone and Brick Chips Building

- From times immemorial, stone has been used both for residential as well as public buildings. Historical buildings that stand today are the living examples of the strength, durability and the excellent weather-resisting qualities of stone masonry. Brick on the other hand has much less strength, durability and weather-resisting qualities.
- On account of its high crushing strength stone is used in the construction of peers, docks, dams and other marine structures. Brick on the other hand, is not considered suitable in all such places.
- Shining texture of good class of stone masonry requires no treatment to enhance its appearance. On the other hand, plastering is necessary to conceal the defects in brick masonry.
- In buildings of monumental nature where architecture requires heavy moldings with large projections, stone is best suited, brick being suitable for light ornamental work.
- Bricks when exposed are liable to get damp. Dampness may ultimately head to the disintegration of the masonry. Stone work on the other hand suffers from no such danger.
- Certain salts present in the sewage react chemically with exposed brick and as such when brick work happens to come in contract with sewage, it is always plastered. Stone on the other hand can be used in such places without providing any protective coat.
- On account of the high cost of stone masonry, its usage is generally restricted to hilly areas or stone districts, bricks on the other hand are easily available in almost at all places and the masonry constructed with bricks costs much less.
- First class bricks possess all such qualities which are required for a good construction and hence brick masonry has now practically replaced stone masonry.
- On account of their regular shape and size, bricks afford great facility in maintaining proper bond in the masonry. It also results in quick construction. On the other hand, in stone masonry, the process of dressing and placing stones requires a great deal of time and extra labor.
- For the construction of jambs of and windows and for the walls meeting at obtuse or acute angle, bricks offer grater facility than stone.
- Bricks cab be conveniently molded into any desired shape at reasonable cost while the expense of the molding of stone work is far more than that of brick
- On account of their convenient size and light weight, bricks require no lifting tackle while in stone masonry the large blocks of stone have to be kept in position with the aid of some lifting device.
- Brickwork is more fire resisting than stone work.

4. METHODOLOGY

In this research, detail construction data are collected and analyzed from 02 different residential building projects situated in Dhaka, Bangladesh. The buildings are so selected that the building plan area and no. of floors remain same for the simplicity of comparison. The life cycle analysis (LCA) of building materials show that the energy requirement and CO2 emission is mainly by two ways: active & passive. Table 1 shows the summary of LCA of building materials with the reasons of CO2 emission & energy consumption from the preparation, transportation to the site and use of these materials.

Table 6: Reasons of CO_2 Emission & Energy Consumption

Sl. No.	Description of Construction Items	Sources of CO_2 Emission & Energy Consumption				
		Wood Cutting	Burning of Wood/Gas/Coal	Electricity for Machine Operation	Plant Operation	Fuel Burning for Transportation
1	Cement		√	√	√	√
2	Brick					
i)	Cutting, Carrying & Mixing of Earth			√		√
ii)	Molding Works				√	
iii)	Burning Sources:					
	Wood	√	√			
	Gas		√			
	Coal		√			
iv)	Kiln Operation & Maintenance				√	
v)	Brick Transportation to Construction Site					√
3	Stone					
i)	Collection of Boulder/Stone Sources					√
ii)	Crushing of Boulder			√		
iii)	Transportation to Construction Site					√
4	Sand				√	√
5	Rebar		√	√	√	√
6	Glass		√	√	√	√

4.1 Estimation of CO_2 Emission, Energy Consumption and cost

We have taken a project and calculated the total material required of 5400 Sft in Dhaka city try to specified the location and summarize all calculation that can be added into it.

Table2:CO_2 Emission & Energy Consumption for 02 Residential Buildings at Dhaka, Bangladesh (construction phase)

Sl. No.	Item Description	Project-1	Project-2	Standard Value Per Unit		CO_2 Emission (Ton)		Energy Consumption (GJ)	
				CO2 Emission (Ton)	Energy Consumption (GJ)	Project-1	Project-2	Project-1	Project-2
	Construction Materials (construction phase)								
1.	Cement(Bags)	12440	7398	0.0194	0.0935	241.34	143.53	1163.2	691.71
2.	Brick (Nos.)	-	845222	0.00054	0.00575	-	456.8	-	4870.6
3.	Stone (Cft)	57258	-	0.00356	0.00483	203.8	-	2765.5	-
4.	Sand (Cft)	28580	18174	0.00138	0.02346	39.5	25	670.5	426.36
5.	Rebar(Kg)	133000	133000	0.0000624	0.001365	8.3	8.3	181.545	181.545
6.	Glass (Kg)	3500	3500	0.0013	0.0184	4.55	4.55	64.4	64.4
7.	Lime(Ton)	3	3	0.47	5.69	1.41	1.41	17.1	17.1
					Total =	498.9	639.59	4,862.25	6,251.72
			Difference			**140.7Tons**		**1389.47GJ**	

Table 3: Total cost analysis of those two Building.

Sl. No.	Item Description	Project-1	Project-2	Per Unit cost (maximum price)	Amount of cost (taka)		Total amount of cost (taka)		Difference
				Cost (Taka)	Project-1	Project-2	Project-1	Project-2	
	Construction Materials (construction phase)								
1.	Cement(Bags)	12440	7398	500	6220000	3699000	27715040 TK	25162120 TK	2552920 TK
2.	Brick (Nos.)	-	845222	10	0	8452220			
3.	Stone (Cft)	57258	-	130	7443540	0			
4.	Sand (Cft)	28580	18174	100	2858000	1817400			
5.	Rebar(Kg)	133000	133000	80	10640000	10640000			
6.	Glass (Kg)	3500	3500	150	525000	525000			
7.	Lime(Ton)	3	3	9500	28500	28500			

5. RESEARCH FINDINGS

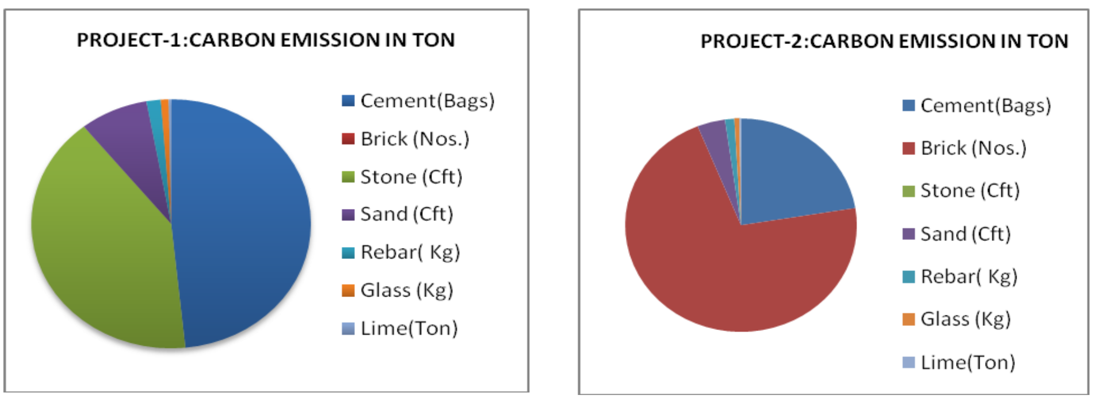
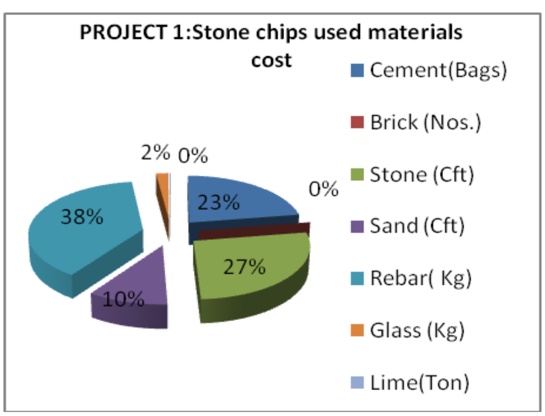


Figure 2: Total carbon emission of the Building.



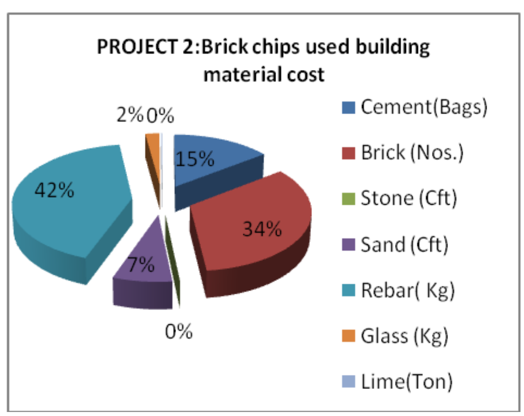


Figure3: Details of building materials cost.

For construction using brick about 639.59 tons carbon-dioxide is emitted and energy consumption is about 6251.72 GJ from a six storied building in construction phase .On the hand for using stones instead of bricks we can reduces carbon-dioxide emission about **140.7Tons** (639.59-498.9) and reduction of energy consumption is about **1389.47GJ** (6251.72-4862.25) of same area of 5400sft . So using stone chips with the environmental point of view is beneficiary for the structures. For brick chips used construction are little cheaper than stone chips used construction. Almost 500 Taka saved per square feet using brick chips than stone chips. So using brick chips with the economical point of view is better than stone chips.

6. CONCLUSIONS

A huge number of concrete buildings are constructed in the world every year, every month. Impact of emission of carbon dioxide is threatened on environment. But we are not Concern. Because, Bangladesh have no data how much CO_2 is released from a building from its construction phase to the end of its life. The use of 120 million MT concrete of which 10 million MT water and 20 million MT cement and 90 million MT aggregate is used in Bangladesh. The total aggregate used in the world is 9 billion tones and it is about 70 % of concrete volume. So it is important aspect in concrete production and its production and transportation will emit carbon dioxide and consume fuel. So by using energy saving material, not sacrificing strength is important and locally available materials induce in concrete will help by the course. The paper only focuses on the building materials used in construction and not on the functions of the building. Initially volume of each building material is estimated and later carbon dioxide emission due to each material is evaluated. It is observed that stone, steel, concrete and Gypsum plaster are the highest energy consumer materials among the all materials used for construction. Finally suggestions are given to reduce the carbon foot print
from a typical building and available materials should be used in construction.

REFERENCES

[1] Bangladesh Bureau of Statistics (BBS) (1991, 1996, 1999), "Statistical Year Book of Bangladesh".

[2] EPA (2008). Life cycle assessment (LCA). Website(http://www.epa.gov/ORD/NRMRL/lcaccess/index.html)

[3]Scheuer, C.; Keoleian, G.A.; Reppe, P. *(*2003). "Life cycle energy and environmental performance of a new university building: modelling challenges design implications".,pp- *35*, 1049-1064.

[4]Junnila, S.; Harvath, A. (2003)Life cycle Envirnmental Effects of an Office Building. *J. Infrastruct. Syst.*, pp-*9*, 157-167.

[5]Gardiner and Theobald, (1990)"Green" buildings: aspects of construction that affect the environment. Research Information, Information Paper 90015, U.K., 16 pp.

[6]Blanchard ,S.,Reppe ,P.(2008). "Life Cycle Analysis Of A Residential Home In Michigan**".**A Project Submitted In Partial Fulfillment Of Requirements For The Degree Of Master Of Science Of Natural Resources.PP95-98

[7]Norman,J. and et.al. (2009) " Comparing High And Low Residential Density: Life-Cycle Analysis Of Energy Use And Greenhouse Gas Emissions". pp-102-105

[8]Levine, M., D. Urge-Vorsatz, et al. (2007). Residential and commercial buildings. In Climate Change 2007: Mitigation. Contribution of Working Group III to the Fourth Assessment Report of the Intergovernmental Panel on Climate Change. USA .pp72-88.

[9]Sagheb,A. and et.al.(2011). "The Role of Building Construction Materials on Global Warming" .National Conference on Recent Trends in Civil Mechanical Engineering .India .pp4-7.

DEVELOPMENT OF A HYBRID WATER TREATMENT SYSTEM USING SOLAR STILL CUM SAND FILTER WITH CERAMIC MEDIA

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ABSTRACT

A vast problem in developing the appropriate water supply system for the communities is that the aquifers containing fresh water are not always found at convenient locations. Especially in coastal areas, there is a much scarcity of pure water because of high salinity in ground water. Again, surface water is being contaminated by various kinds of organisms and compounds. In that case, Desalination is very effective for drinking water supply. This study has been carried out to improve the water quality by using tubular solar still and ceramic media along with sand filtration to provide adequate and safe water supply in coastal areas. In this study, solar desalination with a tubular solar still along with sand filter (TSS-SF) was designed and constructed. The laboratory experiment was carried out on the top roof of the Civil Engineering Department of Khulna University of Engineering and Technology (KUET). The distilled water quality and quantity were properly monitored. Various water quality tests were performed between raw water and treated water. Based on the analysis in study, some proposals are given for the modification of TSS-SF.

Keywords: Aquifer, salinity, desalination, organisms and ceramic media.

1. INTRODUCTION

Water is a basic need for human being for physiological processes. Water demand is increasing day by day due to increasing population. According to World health Organization (WHO), one third of the world's populations are without access of clean drinking water. In 2000, 1.1 billion people worldwide lacked sufficient water resources due to contamination (Sobsey, 2002). Although water is one of the abundant resources on earth but only about 3% of it is potable and remaining 97% is saline water which is lying in the sea. This small percentage of the earth's water which supplies most of human and animal needs exists in ice caps, glaciers ground and surface water source (Ahmed and Rahman, 2000). Groundwater has been used for various purposes such as drinking, domestic purposes, irrigation, industrialization and so on. So, water level is lowering day by day. Surface water is being polluted by various organisms, organic and inorganic compounds. Groundwater also has less exposure to airborne contaminants as well as animal faeces. Generally, groundwater is clear, colourless with very little or no suspended solids and it is free from disease producing micro-organisms which normally present in surface water. For these reasons groundwater is the main source of water supply in urban and rural areas of many developing countries but in some areas groundwater is contaminated with arsenic, and excessive dissolved iron. Besides, in coastal areas groundwater contains higher level of salinity. Solar desalination is the process that removes excess salts and other organic compounds from the water. Though basin type solar still is most popular method in solar desalination, but tubular solar still is easy for construction, operation and maintenance than the basin type solar still. It is the process for water purification where solar energy is used as input energy, raw water is evaporated from the storage channel of solar still and finally accumulated into the outlet. It creates distilled water in undeveloped places that have access only to sea water, brackish or contaminated water.

Among many processes of water purification, filtration is a process in which water is allowed to pass through a bed of filtering media, usually sand and gravel. Among various techniques of filtration slow sand filtration or biological filtration is the process where raw water passes through a bed of sand. Slow sand filtration is a suitable method for water treatment in developing countries because it does not require any complex electrical and mechanical equipment or coagulating chemicals. The major advantages of slow sand filter are it has very high removal turbidity, color and bacteria; cleaning of filter bed by scraping and removal of a top layer of sand and low cost of operation and maintenance. It reduces the number of micro-organisms and other physicochemical compounds present in the water. The fundamental requirements of drinking water are that it should be free from disease producing pathogenic micro-organisms; contain no elements or compound in concentrations that can cause acute or long-term adverse effect on human health; be fairly clear and aesthetically attractive (i.e. low turbidity and color); contain no compounds that can cause an offensive taste and odor; not cause corrosion, scale formation discoloration or staining. There has been a belief that if the technology is simple, low-cost and run by the local community; then it will be effective in promoting a better quality of life. So, the TSS-SF unit could be a good source of producing potable water because of simple construction, locally available materials and semi-skilled or unskilled operators for operations and maintenance. The main objective of this study is to develop a novel water treatment technique using solar still cum sand filter which would increase the production of treated water and retain the salinity within a permissible level.

2. METHODOLOGY

A low cost Tubular Solar Still cum Sand Filter (TSS-SF) was designed and constructed. It was constructed of tubular frame covered with a transparent normal polythene paper and a black rectangular tray used as channel for storing saline water along with sand filter. In the tray, 3 ceramic filters were used as ceramic media for enhancing water surface area. The sand filter consists of a circular filter box and an outlet. The filtration materials were clean and free from clay and gravel. The influent raw water quality in both TSS and SF was remained same. The effluent treated waters from these treatment operations were blended together and stored in a container. The schematic diagram of the TSS-SF unit is shown below:

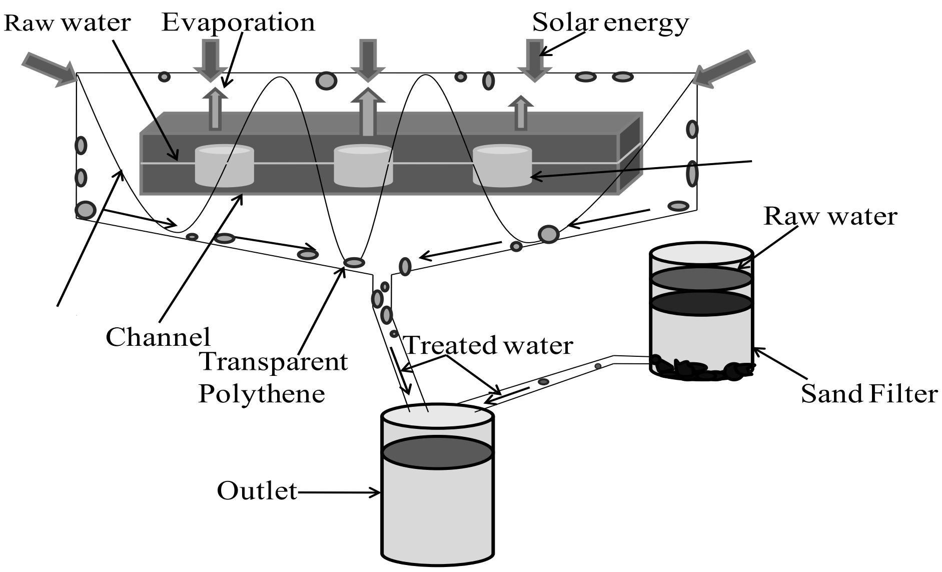

Figure 8: Schematic diagram of TSS-SF unit

2.1 Construction of Channel

A black rectangular tray of length 1.22 m, width 0.25 m and height 0.15 m is used as a channel which is made by cartoon paper wrapping black polythene.

Figure 2: Black rectangular tray

2.2 Construction of Tubular Solar Still

A circular tubular frame of 0.30 m diameter was constructed by using 10 and 14 no GI wire & a tray was used as a channel inside it. It was supported by a wooden stand of 1.22 m length, 0.25 m width and some bricks around the bottom of the stand also with a simple outlet.

Figure 3: Tubular Solar Still (TSS)

2.3 Construction of Sand Filter

A circular slow sand filter was constructed using 8cm thickness of gravel and charcoal layer, thickness of sand layer (sieve 100#, 50#, 30# retain, 16# passing) 55cm, diameter of sand filter 16cm and a simple outlet pipe. The surface area of sand filter 200 sq. cm.

Figure 4: Sand Filter (SF)

2.4 Ceramic Filter

In the tray 3 circular hollow ceramic media are kept. They are about 5 in. in length and 3 in. in diameter. They are used specially to increase water surface area on tray and evaporation effect. It is made by a mixture of sand (80%) and rice husk (20%).

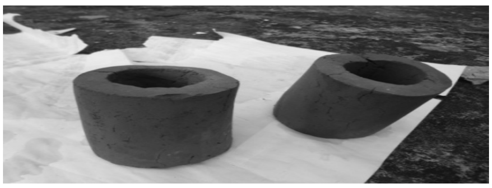

Figure 5: Ceramic Filter

3. RESULTS AND DISCUSSION

3.1 Quality of Treated Water

In winter, salinity value of raw water was found about 1550 mg/L. After treatment, the values of salinity of treated water from TSS, SF and TSS-SF were 0 mg/L, 1481 mg/L and 400 mg/L, respectively (Fig.6). In summer, salinity value of raw water was found to be 2100 mg/L. After treatment, the values of salinity of treated water from TSS, SF and TSS-SF were 0 mg/L, 1983 mg/L and 400 mg/L, respectively (Fig.7). But, the maximum value of salinity in water that is acceptable for drinking is 1000 mg/L. So, the salinity of treated water of TSS-SF unit is within the permissible limit.

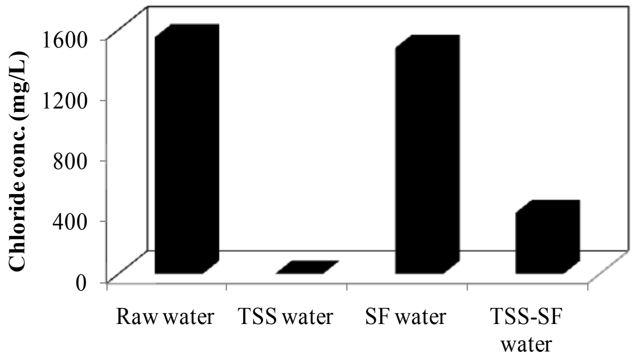

Figure 6: Variation of salinity in winter season

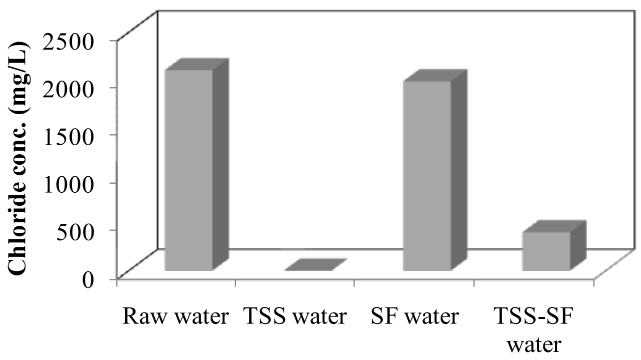

Figure 7: Variation of salinity in summer season

In winter season, the total coliform (TC) of raw water was found to be 22 N/100 ml. After treatment process, the values of total coliform of treated water from TSS, SF and TSS-SF were 0 N/100 ml (Fig. 8). In summer season, the total coliform (TC) of raw water was found to be 21 N/100 ml. After treatment process, the values of total coliform of treated water from TSS, SF and TSS-SF were 0 N/100 ml (Fig. 9). According to WHO guideline (1993) and Bangladesh standards (ECR, 1997), the TC value of drinking water is zero. So, the TC value of treated water is within the permissible limit.

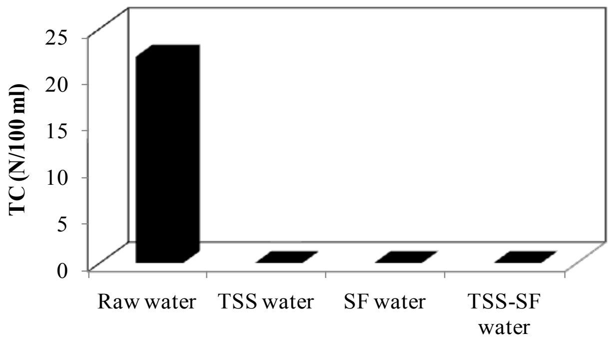

Figure 8: Variation of TC in winter season

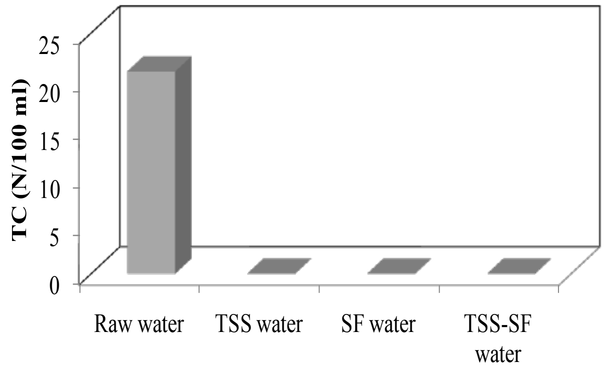

Figure 9: Variation of TC in summer season

In winter season, the total dissolved solids (TDS) value of raw water was found to be 2880 mg/L. After treatment process, the values of total dissolved solids of treated water from TSS, SF and TSS-SF were 0 mg/L, 1745 mg/L and 850 mg/L, respectively (Fig. 10). In summer season, the total dissolved solids (TDS) value of raw water was found to be 3500 mg/L. After treatment process, the values of total dissolved solids of treated water from TSS, SF and TSS-SF were 0 mg/L, 1813.2 mg/L and 868 mg/L, respectively (Fig. 11). According to WHO guideline (1993) and Bangladesh standards (ECR, 1997), the TDS value of drinking water is 1000 mg/L and TDS value of treated water from our TSS-SF was lying below 1000 mg/L. So, the TDS value is within the permissible limit.

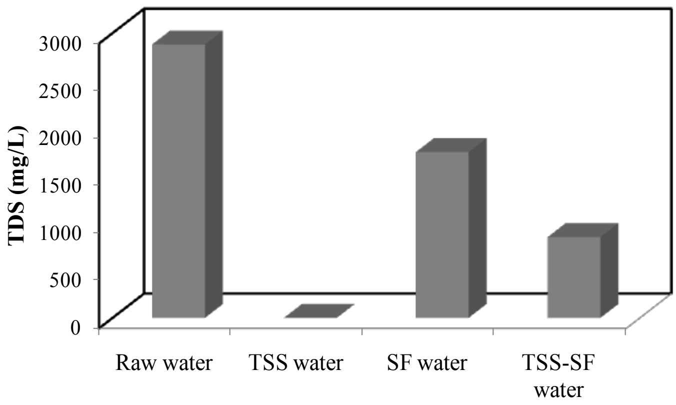

Figure 10: Variation of TDS in winter season

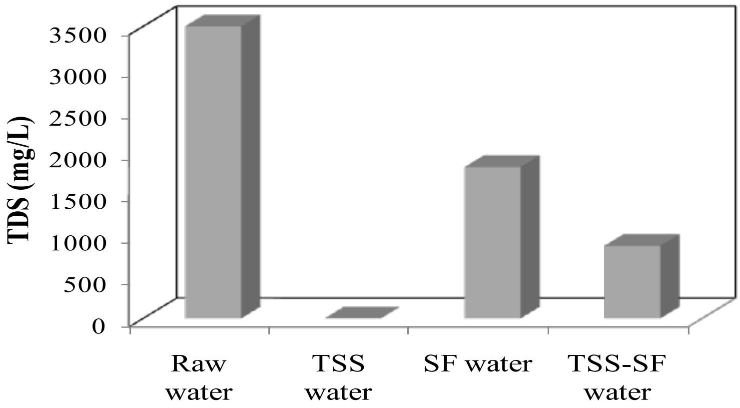

Figure 11: Variation of TDS in summer season

Other water quality parameters such as TS, FC, DO, color and pH of the treated water from TSS-SF unit were within the permissible limit of drinking water quality standards.

3.2 Quantity of Treated Water

In winter, the average daily production rate of TSS was 2.1 L/m^2 and average daily production was 0.65 L. The average daily production rate of TSS-SF was 2.63 L/day. Each seven days total output in winter is shown in Table 1.

Table 7: Treated water from TSS-SF during winter season

Observation Period (Week)	Quantity of Treated Water on TSS-SF (L)
Dec. (19 – 25): 1st	19.2
Dec. 29 – Jan.04: 2nd	18.7
Jan. (6 – 12): 3rd	18
Jan. (13 – 19): 4th	17.6

In summer, the average daily production rate of TSS was 3.15 L/m^2 and average daily production was 0.98 L. The average daily production rate of TSS-SF was 3 L/day. Each seven days total output in summer is shown in Table 2.

Table 8: Treated water from TSS-SF unit during summer season

Observation Period (Week)	Quantity of Treated Water on TSS-SF (L)
Mar. (6 – 12): 1st	20
Mar. (13 – 19): 2nd	20.6
Mar. (20 – 26): 3rd	21.1
Mar. 27 – Apr. 02: 4th	21.7

According to Molla and Biswas (2009), the average daily production rate of distilled water from TSS was 3.54 L/m^2. According to Bokshi and Khatun (2010), the average daily production rate of distilled water from TSS was 2.38 L/m^2. According to Saha (2011), it was 3.04 L/m^2 and according to Rab (2011), it was 1.89 L/m^2 and the average daily production was 0.302 L. According to Raihan (2011), the average daily production rate of distilled water from the inclined basin type solar still (BSS) was 3.78 L/m^2 and the average daily production was 1.85 L. But in this above TSS, the average daily production rate in winter and summer were 2.1 L/m^2 and 3.15 L/m^2 respectively. Average daily production on TSS in both winter and summer were 0.65 L and 0.98 L respectively. According to Sultana and Jesmine (2009), flow rate of slow sand filter was 0.45 m^3 per m^2 per hr. Flow rate of this above sand filter was 0.62 m^3 per m^2 per hr. There was great difference in the production rate because it depends on the design and maintenance. In operation time, some ceramic filters were kept in the tray. Again tray stored about 15 L Water. As a result, a huge amount of load was created upon the tubular frame and the tray. It caused a great amount of bending on the tray as well as the tubular frame (Fig. 12).

Figure 12: Condition of TSS because of heavy loading

This situation would have been removed by inserting a wooden slit into tubular frame. This wooden slit was kept under the tray to support the TSS set-up. This condition is shown in Fig. 13.

Figure 13: Wooden slit is placed under the tray

4. CONCLUSIONS

At present, the interest of potable water is increasing day by day because of industrial progress, intensified agriculture, improvement of standard of life and increase of the population of the world. The supply of pure drinking water is a serious problem for most parts of the world. In summer, there is a serious problem about getting pure drinking water. In coastal areas, there is a much scarcity of pure water because of high salinity in water. Again, surface water is being contaminated by various kinds of organisms and compounds. In this cage, TSS-SF provides a good result for ensuring a good quality of water. The TSS-SF unit is suitable for producing potable water from mostly available saline water in the coastal belt of Bangladesh. Its production cost is also very low. On the other hand, ceramic filter has shown a good result in productivity. The construction of the TSS-SF unit is very easy and one can use this technology easily in the house roof-top. So, it is concluded that the application of TSS-SF technique can fulfill the demand of potable water in the coastal areas of many developing countries like Bangladesh.

REFERENCES

Ahmed, M.Feroze and Rahman, Md. Mujibur (2000).“Water Supply & Sanitation”. ITN-Bangladesh, BUET, Dhaka, pp. 330-450.

Bangladesh Bureau of Statistics (1997), “Statistical Year Book of Bangladesh.” Ministry of Planning Govt. of people’s Republic of Bangladesh, pp. 99-150.

Bokshi, A. and Khatun, H (2010). “Desalination using low cost Tubular solar still” – Thesis report of B.Sc. Program, KUET, pp. 17-30.

Molla, Rabiul and Biswas, Chandra (2009). “Solar Desalination using Low Cost Tubular Solar Still”- Thesis report of B.Sc. program, KUET, pp.10-18.

Rab, Md. Abdur (2011). “Solar Desalination using Low Cost Tubular Solar Still (TSS) with Normal Polythene Paper as Tubular Cover” - Thesis report of B.Sc. Program, KUET, pp.10-40.

Raihan, Abu (2011). “Solar Desalination with inclined Basin Type Solar Still (BSS)”- Thesis report of B.Sc. program, KUET, pp. 19-33.

Saha, A. Kumar (2011). “Solar Desalination using Low Cost Tubular Solar Still” – Thesis report of B.Sc. Program, KUET, pp. 19-25.

Sobsey, M. (2002). “Managing Water in the Home: accelerated Health Gains from Improved Water Supply”. World Health Organization (WHO), North California, USA, pp.30-50.

Sultana, Shammi and Jesmine, Mansura (2009). “Monitoring Bacteriological Quality of Water from Household Taps and Filter” - Thesis report of B.Sc. Program, KUET, pp.16-25.

WHO (1993). “Guidelines for Drinking Water Quality”. 2nd Edition, World Health Organization, Geneva, Swizerland, pp. 60-70.

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URBANIZATION AND SUSTAINABILITY CHALLENGES IN DHAKA CITY, BANGLADESH

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ABSTRACT

Like other developing countries, urbanization in Bangladesh is a growing phenomenon, which is steady in nature but fretfully affects urban sustainability. Despite urban authorities are concerned about this issue, they often fail to assess the problems due to the fact of uncontrollable and unpredictable rural to urban migration, and negligence of urban poor's sustainable living and access to basic services. This paper tries to embrace the issues of urban population growth and consequential challenges of urban sustainability focusing on solid waste managemetn in Dhaka city. This paper is prepared by a qualitative methodology based on secondary data which were collected from different published and unpublished documents, relevant research articles and books. This study indicates the inadequacy of infrastructural services and basic amenities; environmental degradation; traffic jam and accidents; violence and socio-economic insecurity are the major challenges for Dhaka city which are created through rapid urbanization. One of the direct consequences of urbanization of Dhaka city is the increase in solid waste generation, placing a serious threat to the natural resources, and consequently holding back sustainable development. The paper finally concludes providing some strategies that might be helpful to the policy makers in formulating development policies for sustainable urban services.

Keywords: Urbanization, sustainability, good governance, solid waste, Dhaka city

1. INTRODUCTION

Urbanization is now a global phenomenon. The world's urban population reached 2.9 billion in 2000 and is expected to rise to 5 billion by 2030. A great rural-to-urban demographic shift taking place throughout the world is fuelling this urban growth. As a result, the proportion between urban and rural population is steadily tilting towards urban. Only 29% of the world's population lived in urban areas in 1950; this proportion is increased to 47% by 2000, and projected to account for 61% by 2030 (UN, 2004). The overwhelming population growth in urban areas is a complex product of 'pull' and 'push' factors (Savage, 2006). Urbanization, generally, refers to an increasing shift from agrarian to industrial services and distributive occupations (Mandal, 2000). These services and occupational opportunities as a full factor offer many people to migrate to urban areas from rural areas being stimulated by push factors like natural disasters, economic stagnant, and poverty. Dhaliwal (2000) notes these trends in developing countries as a substantial difference from euro-American industrial urbanization. In the context of developing countries, this kind of urbanization processes a dualistic nature of opportunities as well as challenges. Therefore, cities have both positive and negative dimensions. Positively, it is center place of modernization and communication, and engine of a country's economic development. Moreover, cities are the agglomeration of the riches, economic activities, and modern technological advancement and opportunities (Kleniewski, 2006). On the contrary, cities, particularly in developing countries, are now very vulnerable places to live and enjoy quality of life because of environmental problems, rapid growth of urban poor, and terrorism. Many argue that urbanization does not reduce poverty, rather it gives to rise to enormous problems and challenges (Nazem, 2001). However, the world is gradually going to be completely urbanized. Already, more than half of the people live in urban areas. Various reports suggest that the cities of developing countries, particularly, will be facing tremendous challenges of the unpredictable and uncontrollable urbanization which may generate a huge suffering to the people. In Bangladesh, this situation might be more dangerous while overwhelming rural-urban migration is uncontrollable, good governance is rare, and unequal resources distribution is explicitly visible. For example, the urban poor in the city and their informal living in precarious settlements is a big issue of sustainable urban development. Despite the authorities are concerned about this issue, they neglect it in the name of limitation of resources, urban poor's entitlement in the city, and administrative and political difficulties. Therefore, the trend of present urbanization and lacking in good governance pose huge challenges to the future of sustainable city.

One of the consequences of the global urbanization is increasing volume of solid waste. The rising urban population is generating solid waste at an ever-faster rate. It was estimated that about 1.3 billion metric tons of municipal solid waste was generated globally in 1990 (Beede and Bloom, 1995). At present the yearly production of solid waste in the world may be about 1.6 billion metric tons. A considerable amount of money goes into managing such huge volume of solid waste. Industrially developed countries produce large quantities of wastes. On the other hand , developing country generate relatively less solid waste per capita because of their lower purchasing power and consequent lesser consumption (Caricross and Feachem, 1993). This paper attempts to examine this situation drawing upon examples of developing cities in general and Dhaka city of Bangladesh in particular. This paper indicates that inadequacy of infrastructural services, basic amenities and environmental goods, environmental degradation, traffic jam and accidents, violence and socio-economic insecurity are the major challenges for the cities. To analyze these challenges, the paper provides an overview of urbanization of the world and tries to introduce Dhaka city as one of the fastest growing megacities in developing countries and suffering from environmental hazard like managing of solid waste. The paper is based on secondary data which are collected from different published and unpublished documents, relevant research articles and books. With a simple descriptive analytical approach, the paper is organized as following. Section 2 discusses salient features of world urbanization and focused on Dhaka to indicate its position in the list of megacities as well as in Bangladesh. Section 3 indicates major challenges of rapid urbanization and focused on who are the vulnerable in the cities and in which way. In Section 4, focusing on solid waste management of Dhaka city is given. Finally, Section 5 provides the conclusion followed by some brief recommendations of sustainable urban development focusing on solid waste management.

2. HISTORICAL TREND OF URBANIZATION

Dhaliwal (2000) notes that the populations are particularly concentrated in and around major cities of the world. Moreover, according to UN (2000) report, 90% of urban population growth will be in developing countries of Asia, Africa and Latin America. It is also projected that 80% world's cities will be in developing countries (Dhaliwal, 2000). Thus, it obviously shows that the cities of the developing countries will create much more challenges in the days to come.

2.1 World's Urban Population

Cities are currently home to more than half of the world's population. The UN (United Nations) forecasts that today's urban population of 3.2 billion will rise to nearly 5 billion by 2030, when three out of five people will be living in cities (Lewis, 2007). Brockerhoff (2000) notes that a majority of the population of less developed countries will be living in urban areas by 2020, and dramatically in Asia and Africa (IRIN, 2006).

Table 1 shows an increase in urban population in the world from 1950 to 2030. As can be seen in the table, by 2030, 60.3% of the population will be living in urban areas of the world, whereas it was only 29.7% in 1950. Notable, more developed countries show a saturated kind of urbanization, while less developed countries' urban population increases from 17.8 to 39.9% between 1950 and 2000, and will be more than tripled between 1950 and 2030 increasing from 17.8 to 56.2%. The table also depicts that Asia and Africa, particularly, may face tremendous urban pressure in the next several decades. More importantly, many cities of this region will get huge urban population, which are already known as megacities.

Table 1: Urban Population in the World, 1950-2030 (World Urbanization Prospects, 1999)

Region	Percentage of urban population			
	1950	1975	2000	2030
World	29.7	37.9	47.0	60.3
More developed regions	54.9	70.0	76.0	83.5
Less developing regions	17.8	26.8	39.9	56.2
North America	64	74	77	84
Latin America and the Caribbean	41	61	75	83
Europe	52	67	75	83
Oceania	62	72	70	74
Africa	15	25	38	55
Asia	25	37	37	53

2.2 Urban Population Trends in Megacities

A megacity is usually defined as a recognized metropolitan area with a total population in excess of 10 million people. The UN estimates that there were 19 megacities in the world at the beginning of the 21st century

(Brockerhoff, 2000).The City Population (2008) reports that it has already been 26 in number and mostly located in developing countries. Previous reports suggest that there were 83 cities or city systems with populations of more than 1 million in 1950, in which 34 of them were in developing countries. In 2000, this figure rose to 280 with same populations and expected to be double by 2015 (Rodrigue et al., 2006) and just under 500 in 2025 (Dogan and Kasarda 1988; Kasarda and Rondinelli, 1990). All the new millionaire cities are located in the developing countries. Moreover, if cities of 8 million are considered, 28 cities were existed in 2000, in which 22 cities were in developing countries. Despite, in 1950, New York city had only more than 10 million people, it is estimated that by 2015, 12 cities will have a population of more than 15 million (Rodrigue et al., 2006). Table 2 shows 10 largest cities of the world in different times.

Table 2: World's Largest Cities (megacities), 1900-2015 (Brockerhoff, 2000)

Rank	Population (P) in millions							
	1900		1970		2005		2015	
	City	P	City	P	City	P	City	P
1	London	6.5	Tokyo	16.5	Tokyo	35.2	Tokyo	35.5
2	New York	4.2	New York	16.2	Mexico City	19.4	Mumbai	21.9
3	Paris	3.3	Shanghai	11.2	New York	18.7	Mexico City	21.6
4	Berlin	2.7	Osaka	9.4	Sao Pauol	18.3	Sao Paulo	20.5
5	Chicago	1.7	Mexico	9.1	Mumbai	18.2	New York	19.9
6	Vienna	1.7	London	8.6	Delhi	15.0	Delhi	18.6
7	Tokyo	1.5	Paris	8.5	Shanghai	14.5	Shanhai	17.2
8	St. Petersburg	1.4	Buenos Aires	8.4	Kolkata	14.3	Kolkata	17.0
9	Manchester	1.4	Los Angeles	8.4	Jakarta	13.2	Dhaka	16.8
10	Philadelphia	1.4	Beijing	8.1	Buenos Aires	12.6	Jakarta	16.8

As can be seen in the Table 2, the contribution of Asian cities in world's urban population is tremendously increasing. By 2015, 7 Asian cities of 10 will be included in the list of largest cities, while the corresponding figure was 6 in 2005, 4 in 1970, and only 1 in 1900. Moreover, the cities of the developed countries are gradually going off from the list. In 1970, their number was 6, while in 2015, it will be only 2 (Tokyo and New York). Thus, obviously, the major contribution will come from the cities of developing countries. Notably, Dhaka will be included in the list for the first time.

2.3 Urban Population in Bangladesh and Dhaka

Bangladesh is one of the most heavily populated countries in the world. The country is experiencing a rapid urban population growth (13.5 million in 1981, 22.9 million 1990, 37.3 in 2000, and 46.4 in 2005) in recent decades (Chowdhury and Amin, 2006). Uneven development and regional policies, natural hazards, and the lack of employment opportunities in the rural areas are the key factors of urban population growth. Islam (2001a) indicates three factors of rapid urban population growth; (1) a high natural increase in native urban population, (2) the territorial extension of existing urban areas and a change in definition of urban areas, and (3) rural to urban migration. During 1974-1981, Bangladesh experienced higher urban population growth rate at 10.03% due to the facts of both pull factors and push factors (BBS, 2001). The pull factors are employment opportunities, higher wage and income, better life status, educational opportunities, transportation facilities, comparatively better social security etc. On the other contrary, push factors are poverty and lack of employment in rural area; no higher educational institutes; natural calamity like river erosion, cyclones, floods etc. Therefore, migration is the most dominant factor of dramatic urban population growth, and for the large city like Dhaka it is up to 70% (Islam, 2001a). World Bank (2007) notes that 300,000-4000, 000 new poor migrants arrive in Dhaka in a year. The overall population growth in Bangladesh is higher than many other developing countries in the world. In 2005, the estimated population was 152 million in Bangladesh with growth rate of 1.2%, which was 2.6% in 1990. Despite the overall growth rate is decreasing, the urban population growth rate is remarkably high, particularly in the big cities such as Dhaka, Chittagong, Khulna, and Rajshahi. In 1974, only 8.8% of 76 million people lived in urban areas. In 2004, the level of urbanization reached nearly to 25% (BBS, 2005). At present, the overall urban growth rate is steady, but it is huge in few big cities, like in the capital city Dhaka (Islam, 1999). This rapid urban growth plays a pivotal role in the quest for sustainable city, which is very challenging to the government in formulating strategies of urban sustainability (Rana, 2009). The nature of urbanization can be measured considering two components i.e., the level of urbanization and growth rate of urban population. In Bangladesh, the level of urbanization is still low, but its total urban population is very large, 28.60 million in 2001. Table 3 shows that the proportion of population has increased tremendously from only 2.54% (1911) to nearly 23% in 2001. Before Pakistan period, the urban population growth was steady (about 3%). But, it experienced a tremendous growth (4.34% in 1951-8.89% in 1974) in Pakistan period, which continued up to first two decade of Bangladesh period (20.15% in 1991). The annual growth rate was abruptly high (10.03%) during 1974-1981 because of rural-urban migration as the result of huge famine in many remote village areas of the country. During 1981-1991 periods, a slower growth of urban population (5.43%) was observed in comparison with the previous decade. During 1991-2001, the growth further declined to 3.15%, but still remained much higher than the national population growth rate. The overall growth indicates that the urban population in the country has been doubled every 12 years (CUS, 2001).

Table 3: Growth of Urban Population in Bangladesh, 1991-2001 (BBS, 2001)

Census Year	Total national population (million)	Annual growth rate (percent)	Total urban population (million)	Percent of urban population	Decadal increase of urban population (percent)	Annual exceptional growth rate of urban population (percent)
1901	28.2	0.70	2.43	-	-	-
1911	31.65	0.94	0.80	2.54	14.96	1.39
1922	33.25	0.60	0.87	2.61	8.85	0.84
1931	35.60	0.74	1.07	3.01	22.20	2.00
1941	41.99	1.70	1.54	3.66	43.20	3.59
1951	44.17	0.50	1.83	4.34	18.38	1.58
1961	55.22	2.26	2.64	5.14	45.11	3.72
1974	76.37	2.48	6.00	8.89	137.57	6.62
1981	89.91	2.32	13.56	15.54	110.68	10.03
1991	111.45	2.17	22.45	20.15	69.75	5.43
2001	123.85	1.54	28.60	23.01	37.04	3.15

3. SUSTAINABILITY RELATED PROBLEM IN BANGLADESH: MAJOR CHALLENGES

Urbanization has brought remarkable development in Bangladesh, even though it has been a great challenge environmentally, socially, and economically. To build a sustainable city, these challenges need to be faced efficiently and successfully. In general, a sustainable city must be economically viable, socially peaceful, and environmentally friendly. More specially, a sustainable city is where people live in peace with sufficient income earning and quality of life, and without social and mental anxiety.

Hardoy et al. (1992) note that a sustainable city provides healthy environment and meets multiple goals i.e., healthy living and working environments: access to water and sanitation, waste disposal, drains, paved roads, and other forms of infrastructure and services essential for health and for a prosperous socioeconomic base. These definitions indicate major challenges of urbanizations as well as characteristics of a sustainable city. Drawing upon these definitions, the following sections try to describe major challenges, particularly in Dhaka city, which are very important for sustainable urban development.

3.1 Environmental Problems

Environmental problems in the cities constitute air, water, and noise pollution, and also problem related with solid wastes (toxic or hazardous wastes). The process of industrialization and urbanization leads to deterioration of healthy environmental conditions. Uses of fossil fuels in industry, transportation, and household cause huge contamination of air, water, and soil. For example, 12.60% of the death in Jakarta is related to air pollution causes (World Resources Institute, 1996). Notable, traffic-generated pollutants such as particulates, nitrogen oxide, and carbon dioxide have been increasing in the air of the cities. Moreover, the environmental problems are particularly serious in Third World Cities because of industrialization and urbanization in one or two cities of the country. For instance, Bangkok, Dhaka, Mexico City, and Sao Paulo include a high proportion of industrial output. These cities are particularly affected by these environmental problems and suffer comparable industrial problems to those in Europe, Japan, and North America. In many cities, environmental problems are far serious (Hardoy et al., 1992). Rapid urbanization is always characterized by spatial extension in the fringe area, which converts agricultural land into urban area. Large areas around cities are dug up for making bricks, and forests are destroyed to meet the firewood needs (Dhaiwal, 2000). The situation has been particularly dangerous in Dhaka city. Some unscrupulous developers (both corporate and individual) have been involved in grabbing of common pool resources such as water bodies, forests, lakes, and rivers. River Buriganga, the lifeline of Dhaka, has been encroached at almost all areas, thus shrinking its width and navigability (Islam, 2000).

The air quality of Dhaka has been one of the worst among many developing cities in the world. There are two major sources of air pollution which include industrial emissions and vehicular emissions. The industrial sources include brick kilns, fertilizers factories, spinning mills, tanneries, garments, bread and biscuit factories, chemical and pharmaceutical industries, metal workshops etc. The vehicular emissions are caused by two-stroke three wheelers (scooter), poorly maintained old trucks, buses, and other motor vehicles. Bricks kilns around the city also constitute to the worsening of air quality. Recent ban on the use of two-stroke vehicles in Dhaka remarkably contributed to improvement of environmental quality. A recent survey by the department of environment shows that particle matter in the air of Dhaka city was 30% lower in January, 2003 compared with December, 2002 (CPD, 2003). Nonetheless, a study conducted by Bangladesh Atomic Energy Commission reports that about 50 tons of lead is emitted into Dhaka's air annually, and the emission reaches its highest level in dry season (November-January). The density of lead in the air of Dhaka city in dry season reaches 463 monograms per cubic meter, the highest in the world, while it is 383 monograms per cubic meter in Maxico City, and 360 monograms per cubic meter in Mumbai, India (Mahadi, 2010). Water logging is a common feature in Dhaka city. City life becomes hostage to water logging due to lack of proper sewerage management, particularly during rainy season. Rainfall of above 10 mm in 24 h creates water logging in various parts of the city. It is suggested that at least 15% of the total land of the city should be water body in order to maintain a healthy environment. Nevertheless, most of the water body in and around the city has been filled up by illegal construction such as industries, housing, and commercial infrastructure. Earlier, these water bodies were the sources of water supply, rainwater catchment, fish cultivation, open spaces etc. Murtaza (2004) notes that many water bodies in the city have been polluted by a large number of industries, mills, and factories established in an unplanned manner. He further argues that the issues of the water disposal are not considered during setting up these industries. The standard of solid waste management in Dhaka city is also very poor. Islam (1993) argues that municipalities and pourashavas which have responsibilities for collection and disposal of urban wastes do not have the resources to deal with the situation. Daily household wastes are normally thrown out to the roads or open places, which causes water and soil pollution. Industries like tanneries discharges huge waste into the river without any recycling treatment. So, the environmental degradation of the urban areas today is a result of the lack of policy support for adequate investment in urban development and improve urban management (Shafi, 2003), and also for lack of environmental awareness of the people.

3.2 Infrastructure and Services

Rehabilitation of the urban poor and housing are the major challenges in cities of developing country. In some cities (e.g., Mumbai), informal settlements and slumps, and squatters may form more than 50% of cities' population (Islam, 2001b). Already over 90% of the urban population of Ethiopia, Malawi, and Uganda, three of the world's most urban population, were living in informal settlements or slums. Notably, by 2030, the number of the world wide slum dwellers is projected to reach two billion (State of World Population, 2007). Similarly, in Dhaka city of Bangladesh, almost 34% of the city's 13 million residents live in 5,000 slum and squatter settlements (CUS, 2006). The growth of the cities both in terms of areas and population has consistently been faster than the growth of infrastructural provisions and services in Dhaka. As a result, a large section of the urban population does not have access to basic infrastructure services. The services include housing, water and sanitation, drainage, roads, gas, and electricity supply. Insufficient and unhealthy housing in the cities, particularly in Dhaka, is the major infrastructural problem in Bangladesh (Uddin and Jones, 2000). The rapid growth of population and low affordability of the people have resulted in poor housing situations, such as slums and squatters (Islam, 2000). Nearly half of the populations in major cities live in slum and squatter settlements with extremely poor physical conditions (Islam and Hasan, 2004). These inadequacies cause a higher mortality rate in the informal community. For example, infant mortality (80 per 1000 live births) and age under-5 mortality rates (140 per 1,000 live births) in urban slums are among the highest in the world (EPHD, 2010). The availability of land for housing is limited, and hence the costs are gradually soaring. Moreover, nearly, two-third of the urban population is unable to acquire and within reasonable distance from their work place (Islam, 1993). Besides housing, the basic utility services like water, electricity, and gas are very essential in everyday life. But, the urban authorities are unable to provide these services to all urban people, particularly to the urban poor. Only 40% of urban populations have access to hygienic sanitation (Islam, 1993). Water supply system is also insufficient for which the quality and quantity of water are always unsatisfactory to the dwellers. The garbage disposal system in the city is yet to be completely satisfactory too. Few areas of the city are going to be managed by sustainable garbage disposal system after government and non-government organizations and community development. Nonetheless, 35.1% of them do not have any garbage collection system. In addition, a study by Enayetullah (1995) reports that the Dhaka city corporation area generates 3,000-3,500 metric tons of municipal solid waste daily, of which 42% is collected by the management authorities, but rest of them remains on roads, open drains , and low-lying areas.

3.3 Poverty and Social Insecurity

The poverty incidence in third world cities suggests that urban is a habitat of extreme opportunities for the riches, but has been worse reality to the poor. It is generally assumed that urban poverty levels are lower than rural poverty levels, but the absolute number of poor and undernourished is rapidly increasing in the cities. Poverty has been considered as a key driver of violent crime too. One billion people, about one-sixth of the world population, now live in shanty towns (Whitehouse, 2005) which are seen as “breeding grounds” for social problems such as crime, drug addiction, alcoholism, poverty, and unemployment. A World Bank study on violence in Latin American Urban areas showed that homicide rates ranged from 6.4 per year per 100,000 in Buenos Aires to 248 in Medellin, Colommbia. Rio de Janerio, Sao Paulo, Maxico City, Lima and Caracas account for more than half of their countries’ nation homicides (IRIN, 2006). In Bangladesh, similarly, slumps and squatter are always considered as the breeding grounds of anti-social elements. It is true that most of the miscreants, hijackers, murderers, and drug suppliers live in this area. That is why, the ever-increasing number of slumps people and their activities has become a non-stop social problem for the cities. Although because of government and non-government interventions, the incidences of poverty has been reduced significantly in the urban areas since after liberation, it is still remains pretty high at nearly 37.9% of total urban population in 2004. This may offset the Milleneium Development Goals of reducing the level of poverty in the country by 50% by the year 2015 (CPD, 2003).

3.4 Natural Man-Induced Hazards

Flood is also a common hazard in the cities of Bangladesh. Besides natural causes, unplanned infrastructure development, and inadequate and inefficient drainage are the main causes of floods. Diminishing water bodies and interrupting river flows are also conducive factors of worsening the situations. As for example, once there were number of canals in Dhaka city, which are now filled up for settlements and business purposes. The Dhaka city protection embankment that was built spending huge resources is not properly maintained by the authority. A lot of urban poor have built houses and shops on the embankment, which gradually damaging its strengths and capacity (Islam, 1998). In 2004, almost 2.5 million people were affected by flood, and 20 people had died (Islam and Hasan, 2004). CUS (2006) reports that more than 60% slums in the city were affected by floods, in which 38.5% were fully affected. Alam and Rabbani (2007) also state sufferings and extent of 1998 flood in Dhaka city which caused extensive water logging. Bangladesh is also susceptible to damaging earthquakes. The city of Chittagong, Mymensing, Rangpur, sylhet, and Dhaka are very prone to earthquake hazards (Ansary, 2004). According to report published by United Nations IDNDR-RADIUS Initiative, Dhaka and Tehran are the cities with the highest relative earthquake disaster risk (Ansary, 2004; Cardona et al., 1999). Depending on the time of the day, between 45,000 and 86,000 people may perish due to collapse and damage of structure. The number of serious injuries may range between 110,000 and 210,000, with severe damage on the emergency relief and healthcare infrastructure (Ansary, 2004).

4. SOLID WASTE AND SOLID WASTE MANAGEMENT

Solid waste is usually categorised into municipal solid waste and industrial waste according to its sources. Solid waste is useless, unwanted, and discarded non-liquid waste materials arising from domestic, trade, commercial, industrial, agriculture as well as public services (Wadood, 1994). Nunan (2000) refers to solid waste management as the collection, treatment, and disposal of municipal solid waste.

4.1 Solid Waste Generation, Population and GDP

The production of urban solid waste is generally increasing in Bangladesh. In 1995, urban Bangladesh generated 0.49 kg/person/day waste which is estimated to increase to 0.6 kg by 2025 due to rapid and high growth of urban population (Ray, 2008). For example, in 1999, 30 million people, around 20% of the total population of Bangladesh, lived in urban areas; by 2015 it is estimated that 68 million, more than one third of total population, will be living in urban areas (Pryer, 2003). Table 4 shows a relation between per capita GDP, population and waste generation in urban areas of Bangladesh. The table demonstrates that increase in per capita income improves the purchasing capacity of citizens, which accelerates the growth of solid waste production.

Table 4: Relationship Between GDP, Population and Waste Generation (Enayetullah and Hashimi, 2006)

Year	Urban population	Total urban waste generation (ton/day)	Per capita waste generation rate in urban areas (kg/cap/day)	Per capita GDP
1991	20.8 million	6493	0.31	US$ 220
2005	32.76 million	13,330	0.41	US$ 482
2025	78.44 million	47,000	0.60	---

4.2 Solid Waste Management

The trend of urban population growth has outstripped the capacity of city governments to provide effective and efficient delivery of conservancy services. As a result, nearly 50% of the daily generated garbage remains uncollected in the cities of Bangladesh (Bhuiyan, 2005). A 'gap' exists between the daily generation and the collection of solid waste, which leaves urban administration vulnerable to citizens' complaints. Weak institutional incentives and capacity have been at the base of the dysfunctional in urban service deliver. The challenge will be even greater by 2020 when the demand for removal of solids waste may raise to 50,000 tons/day from the present 10,000 tons/day (World Bank, 2003). The projected demand for solid waste management (SWM) is indicated in Figure 1.

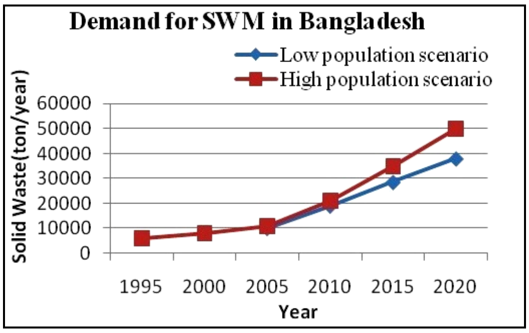

Figure 9: Projected demand for solid waste management in Bangladesh (World Bank, 2003)

4.3 Solid Waste Disposal Problem in Dhaka

Dhaka has already reached the rank of a mega-city with over 10 million populations. The city covers an area of 460 km^2. Cities like Dhaka are confronting a twin dilemma. The urban population is growing rapidly causing a huge increase in demand for waste management services. On the other hand, the traditional public sector is responding poorly to the growing demand for such services (Ahmed and Ali, 2006). Thus, solid waste management has become a major concern for Dhaka where Dhaka City Corporation can pick up and dispose only 42% of the solid waste generated (Salequzzaman et al., 1998). Practically, the city corporation has failed to manage the solid waste of the increasing population, mainly because of lack of financial support and willingness to pay for overall sustainable solid waste management policies. The Dhaka city government was revamped, under a Municipal Ordinance, as Dhaka Municipal Corporation in 1978. In 1990, it was renamed Dhaka City Corporation (DCC) through a Local Government Act. The responsibility of solid waste management is placed on the Conservancy Department of DCC. A Chief Conservancy Officer heads this department. This position was traditionally neglected although solid waste disposal is one of the most important functions of DCC.

4.4 Work process of the Conservancy Department of Dhaka

Dhaka has been divided into 10 administrative zones where a conservancy officer runs the conservancy service of a zone. All wards receive regualar conservancy services. So Dhaka does not have any non-conservacy ward like other municipalties. All service users pay 2% conservancy tax with their annual household taxes.The corporation employs around 7500 sweepers to sweep roads/lanes and clean drains (Bhuiyan, 2001; Paul, 1991). Formally, the cleaners (mainly the female sweepers) of Dhaka are supposed to sweep the roads and lanes daily in three shifts: morning, day, and night within the officially assigned duty hours between 6 am to 2 pm, and 7 pm to 3 am (Asaduzzaman and Hye, 1998). Usualy garbage removal to the final dumping grounds is done at night. One probable reason for night trafficking of garbage is that there is a ban on running any truck (except the trucks used for emergency service duty) in the city during daytime to avoid traffic jams and accidents caused by reckless driving of traffic drivers. There are four dumping grounds in Dhaka: (a) Matuali, (b) Gabtoli, (c) Amin Bazar, and (d) Badda (Bhuiyan, 2010). According to Asaduzzaman and Hye (1998), the system of solid waste collection in Dhaka may be described as a 'dustbin-based' (including demountable containers) collection system. Dustbins together with demountable containers account for about 66% of solid waste, followed by encloses (15%), dust-shoot on street collection (10%), and block collection (9%). Door to door collection system covers only a negligible volume of the waste (0.1%).

4.5 Operational Challenges of Conservancy Department

The conservancy department of Dhaka faces operational challenges such as the lack/misuse of resources, corruption, political interference, central-local government relationships, lack of interdepartmental coordination, and lack of people awareness. Some of these issues are in some detail discussed here. Kandakar (1995) observes that for satisfactory cleaning of a city area at least two sweepers are required per thousand populations. According to this calculation, Dhaka delivers services to their clients with inadequate workforce. But it is argued that if the sweepers perform their task appropriately the magnitude of the problem could be largely reduced. The tendency of a large number of sweepers is to draw their salary without doing any meaningful work. The conservancy staff of Dhaka, particularly those employed on contract basis receives their emoluments without work (Bhuiyan, 2005). This case also reveals the lack of functional accountability in the corporations' work. In order to run an organization effectively, material resources are as important as human resources. The insufficiency of the quantity of material resources is also a barrier for Dhaka city to provide services to a huge population living in the city.

Table 5: Budgetary Allocations for the Conservancy Department, 1996-2001(Bhuiyan, 2005)

Financial Year	Dhaka city (Taka in million)		
	Conservancy	Total budget	% of total budget
2000-2001	140	1129	8.1
1999-2000	135	997	7.4
1998-1999	100	991	9.9
1997-1998	90	996	11.1

Despite the presence of human and material resources, organizational objectives cannot be achieved without adequate financial guarantees. The budgetary allocation for the financial years 1996-2001 is presented in Table 5 to highlight the position of conservancy budget in the context of total (revenue) budgets of Dhaka. An analysis of the financial statements reveals that about 70-80% of the budget allocated for the Conservancy Department was spent for the payment of salary and other fringe benefits to its employees, leaving an insignificant amount of money for the institutional development of the department. This contributes to a better understanding how the city government leaders perceive solid waste as a potential urban problem associated with urban governance. Solid waste has become a matter of global concern as is evident from the conclusion drawn in the colloquium of mayors held at the United Nations in New York in August 1994, where they identified twelve several urban problems, and the problem of solid waste management is ranked third (Islam, 1999). Urban dwellers of Bangladesh are not quite aware of this problem. As a result, they throw garbage randomly instead of properly disposing it into designated bins. The role of central as well as urban government is responsible for this as they largely failed to initiate effective motivational campaigns to increase awareness of people on this issue.

5. CONCLUSION AND RECOMMENDATIONS

Rapid urbanization in the cities of developing countries has been a dilemma of economic development and environmental sufferings. This paper tries to emphasize the issue of urban population growth and consequential challenges of urban sustainability focusing on solid waste managemetn in Dhaka city. It is evident that Dhaka city is overwhelmingly growing because both of the pull factors and push factors, while the service provisions and income earning opportunities are not provided with a same pace. Thus, the city is gradually going to be suffering from inadequate infrastructural services, social insecurity, natural and man-made hazards, and poor urban governance. This study demonstrates that responsible authorities have failed to manage generated solid waste, one of the direct consequences of urbanization in Dhaka city, because of large population and budget deficit. Finally, based on the urban challenges, this paper briefly recommends some strategies that need to be considered in sustainable urban development policies. The recommendations are following:

i. Good governance, an efficient approach in sustainable urban development, implies adequate transparency, accountability, decentralization, participation, coordination, and control which Bangladesh is lacking in. Decentralization of economic growth and decentralization in political administrative structure are inevitable. Dhaka has already grown to a massive population. Thus, immediate efforts should be made to ease the pressure by getting up new satellite cities around the metropolis. Local government or municipal authority should be independent and responsible for urban development and planning instead of central government interference such as budgets. The central government should provide overall master plan and guidelines of the development facilitating services, resources, and funds.

ii. A strong and active body of civil society can stimulate local as well as central government policies for the sake of betterment of the society and sustainability. Environmental and social equity issues need to be given more emphasis in urban development. CUS (Center for Urban Studies), BELA (Bangladesh Environmental Movement Lawyer Association) and BAPA (Bangladesh Poribesh Andolon- Bangladesh Environment Movement) are working in Dhaka with a vision of sustainable city. These organizations need to be supported and strengthened further recognizing formally their existence.

iii. Privatization of the amenity services is another possible action which might be helpful to sustainable urban development. There are lots of success stories of privatization of urban amenities in the developing countries (Stiles, 2002). But, the opposite views of privatization are also significant and considerable because of pro-rich nature of functioning and exclusion of the poor. In Bangladesh, this system might not be successful while a major portion of urban people are absolutely poor.

iv. Increases of monitoring in service quality, particularly in the public sector.

v. Door-to-door garbage collection in particular time notified in advance so that an environmental friendly situation is obtained in the neighborhood.

vi. It is proposed that the solution lies in the private sector participation in delivering solid waste management services. Particularly, public-private partnership (PPP) is often viewed as a potential alternative to the traditional services delivered by the public sector alone.

vii. To ensure that waste is properly segregated at the point of its generation that does not subsequently become mixed with, proper and secure storage facility for hazardous waste followed by appropriate safe final disposal.

viii. Promotion of public awareness, appropriate policy and laws, and willingness are essential for proper solid waste management in Dhaka city.

ix. Finally, we need to accentuate the important and essential issues based on priorities of the major part of the urban people. Moreover, administrative and institutional reform, balanced regional development strategies, and bottom-up approach in development policies should be implemented in on time.

REFERENCES

Ahmed, S. A. and Ali, S. M. (2006). "People as partners: facilitating people's participation in public-private partnerships for solid waste management." Habitat International, Vol. 30(4), 781-796.

Alam, M. and Rabbani, M.D.G. (2007). "Vulnerabilities and responses to climate change for Dhaka." Environment and urbanization, Vol. 19(1), 81-97.

Ansary, M. A. (2004). "Seismic risk in urban areas of Bangladesh." People's Report 2003: Bangladesh Environment. Unnayan Shamannay, Ministry of Environment and Forest, and UNDP.

Asaduzzaman, M. and Hye, H. A. (1998). "When both market and state fail: the crisis of solid waste management in urban Bangladesh." In R. Sobhan (Ed.), Crisis in governance: A review of Bangladesh development 1997, Dhaka: Centre for Policy Dialogue and University Press.

BBS (2005). Bangladesh population census. Bangladesh Bureau of Statistics.

Beede, D. N. and Bloom, D. E. (1995). "The economics of municipal solid waste." World Bank Research Observe, Vol. 10(2).

Bhuiyan, S. H. (2010). "A crisis in governance: Urban solid waste management in Bangladesh." Habitat International, Vol. 34, 125-133.

Brockerhoff, M. P. (2000). An urbanizing world. Population bulletin, 55(3), Population reference bureau.

Cairncross, S. and Feachem, R. G. (1993). Environmental health engineering in the tropics: an introductory text. 2nd edition, John Wiley and Sons, UK.

Cardona, C., Davidson, R. and Villacis, C. (1999). Understanding urban seismic risk around the World. A summary report on the comparative study of the United Nations international decade for natural disaster reduction, RADIUS initiative.

CUS (2001). National policy forum task force on urban governance. CUS Bulletin on Urbanization and Development, No. 41, pp. 2-9, Center for Urban Studies, Dhaka.

CUS (2006). Slums of Urban Bangladesh: Mapping and Census, 2005. Dhaka, Bangladesh.

Chowdhury, F. J. and Amin, A. T. M. N. (2006). "Environmental assessment in slum improvement programs: some evidence from a study on infrastructure projects in two Dhaka slumps." Environmental Impact Assessment Review, Vol. 26, 530-552.

CPD (2003). Urban governance in Bangladesh. CUS Bulletin on Urbanization and Development, No. 44. Center for Policy Dialogue, Dhaka, Bangladesh.

Dhaliwal, C. (2000). Fundamental of environmental science. Kalyani Publishers, India.

Enayetullah, I. (1995). "A study of solid waste management for environmental improvement of Dhaka city." An unpublished Masters of Urban and Regional Planning Thesis, BUET, Dhaka.

Enayetullah, I. and Hashimi, Q. S. I. (2006). "Community based solid waste management through public-private community partnerships: experience of waste concern in Bangladesh." 3rd Asia Conference, Japan.

EPHD (2010). Enviroprotection and human development, Dhaka, Bangladesh, Accessed May 20, 2010, http://www.angelfire.com/ak/EPHE.index.html

Hardoy, J. E., Mitlin, D. and Stterhwaite, D. (1992). Environmental problem in Third World cities. Earthscan Publications Ltd, London.

IRIN (2006). Nigeria: Lagos, the mega-city of slums. UN office for the co-ordination of humanitarian affairs. Accesses February 8, 2008, http://www.energypublisher.com/article.asp?id=5307.

Islam, N. (1993). The Bangladesh situation in urban management capacity. CUS Bulletin on Urbanization and Development. No. 24 and 25, Centre for Urban Studies, Dhaka, Bangladesh.

Islam, N. (1998). "Geo-environment controls of urban development." In international seminar on geo-science and urban development held on 24-26, February 1998, Bangladesh Geographic Society in cooperation with the Association of Geoscientists for International Development, Dhaka, Bangladesh.
Islam, N. (1999). The urban future; millennium thoughts. CUS Bulletin on Urbanization and Development, No. 37. Center for Urban Studies, Dhaka, Bangladesh.
Islam, N. (2000). "Settlements of the urban poor in Bangladesh." US Bulletin on Urbanization and Development, No. 38 (pp. 2-4). Center for Urban Studies, Dhaka, Bangladesh.
Islam, N. (2001a). Urbanization, urban planning and development and urban governance: a reader for students. Centre for Urban Studies, Dhaka, Bangladesh.
Islam, N. (2001b). Good urban governance: an issue reemphasized (editorial notes). CUS Bulletin on Urbanization and Development, No. 41. Centre for Urban Studies, Dhaka, Bangladesh.
Islam, N. and Hassan, S.R. (2004). Flood in Bangladesh 2004. CUS Bulletin on Urbanization and Development, No. 46. Center for Urban Studies, Dhaka, Bangladesh.
Khandakar, M. A. (1995). "Role of Chittagong and Khulna City Corporation in solid waste management." Journal of Local Government, 24(1).
Kleniewski, N. (2006). Cities, change and conflict: a political economy of urban life Australia.
Lewis, M. (2007). Megacities of the future. Accessed February 8, 2008, http://www.forbes.com/2007/o6/11/megacities-population-urbanization-biz-x_21cities_ml_0611megacities.
Mahadi, M. (2010). Air pollution in Dhaka. Accessed May 20, 2010, http://www.scribd.com/doc/10304145/-Air-Pollution-in-Dhaka-City.
Mandal, R. B. (2000). Urban geography: a textbook, Concept Publishing Company, New Delhi, India.
Murtaza, M. G. (2004). Urban disasters in Bangladesh: Some suggestions and concern. CUS Bulletin on Urbanization and Development, No.46, Center for Urabn Studies, Dhaka.
Nazem, N. I. (2001). "The concepts of urbanization and development." In N. Islam (Ed.), Urbanization, urban planning and development and urban governance: a reader for students. Center for Urban Studies, Dhaka.
Nunan, F. (2000). "Urabn organic waste markets: responding to change in Hubli-Dharard, India." Journal of Habitat International, Vol 24(3), 347-360.
Paul, K. (1991). "A note on the solid waste management of Dhaka City Corporation." Journal of local government, 20(2).
Pryer, J. A. (2003). Poverty and vulnerability in Dhaka slums: The urban livelihood study. Ashgate, Aldershot.
Rana, M. M. P. (2009). "Sustainable city in the global North and South: goal or principle." Management of Environmental Quality: An International Journal, Vol. 20(5), 506-521.
Rodrigue, J.P., Comtois, C. and Slack, B. (2006). The geography of transport system. New York: Routledge. Accessed February 12, 2008, http://people.hofstra.edu/geotrans/.
Ray, A. (2008). "Waste management in developing Asia: can trade and co-operation help?" Journal of Environment and Development, Vol. 17(1), 3-25.
Salequzzaman, M., Murtaza, M.G., and Soaroar, M. (1998). Evaluation study on municipal solid waste management project in Khulna City. PRODIPAN, Shaheb Bari Road, Khulna, Bangladesh.
Savage, V. R. (2006). "Ecology matters: sustainable development in Southeast Asia." Journal of Sustainability Science, Vol. 1(1), 37-63.
Shafi, S. A. (2003). Use of planning tools as a guide to balanced urban development. CUS Bulletin on Urbanization and development, No. 44, Center for Urban Studies, Dhaka, Bangladesh.
State of World Population (2007). Accessed February 8, 2008. http://www.unfpa.org/swp/2007/english/ chapter_2/slums.html.
Stiles, K. (2002). "International support for NGOs in Bangladesh: some unintended consequences." World Development, Vol. 30(5), 835-846.
Uddin, M. N. and Jones, E. M. (2000). "Sanitation: the crisis of the urban poor." In 26th WEDC conference on water, sanitation and hygiene: Challenges of the millennium. Dhaka, Bangladesh.
UN (2000). World urbanization prospects: The 1999 revision. New York: United Nations, Population Division.
UN (2004).World urbanization prospects: The 2003 revision. New York: United Nations, Population Division.
Wadood, A. (1994). "Effects of urban waste on environment and its control." In M. Hoq and H. Lechner (Eds.), Aspects of solid waste management: Bangladesh context. Goethe Institute, Dhaka.
Whitehouse, D. (2005). Half of humanity set to go urban, 2005. Accessed February 8, 2008, http://news.bbc.co.uk/hi/science/nature/4561183.stm.
World Bank (2003). Bangladesh 2020: A long-run perspective study. Dhaka university press limited.
World Bank (2007). Dhaka: Improving living conditions for the urban poor. Bangladesh development series, paper no. 17, The World Bank Office, Dhaka.
World Resources Institute (1996). World resources, 1996-97. Oxford University Press, New York.
World urbanization Prospects (1999). Revision: key findings. Accessed June 5, 2008, http://www.un.org/esa/populations/wup1999/wup99.htm.

TREND ANALYSIS OF CLIMATIC EXTREME EVENTS FOR THE HAOR AREA OF BANGLADESH USING CLIMATIC INDICES

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ABSTRACT

The main objective of this study was to obtain the extreme event analysis of trends in nine country relevant annual extreme indices for all the seven Haor districts (Sunamganj, Habiganj, Maulvibazar, Netrakona, Kishorgonj, Sylhet, Brahmanbaria) of Bangladesh. The analysis have been obtained for four BMD metrological stations (Sylhet, Srimongal, Mymensing and Comilla) with a historical time span of near about 50 years (1960 to 2008) to represent the whole Haor area. The software used to process the analysis was the RClimdex 1.0. Long term daily rainfall and temperature (maximum and minimum) data have been used to prepare the input dataset for the software and data has been quality controlled and tested for homogeneity, before they were used for calculation of indices. The analysis has identified that the temperature has increased significantly for some extreme indices (SU25) in Haor area by the last half of the century. Moreover other climatic indices showed that the rainfall intensity in Haor area was also increasing for last 50 years.

Keywords: Change, Extreme Events, Climatic Indices, RClimdex, Trend Analysis, Haor

1. INTRODUCTION

Climate is changing. The change in the global climate is the resultant of the anthropogenic as well as natural events that induced an alteration in the frequency of some climatic extreme events is likely to change (IPCC, 2007; Alexander et al., 2007). Folland et al. (2001) showed that in some regions both temperature and precipitation extremes have already shown amplified responses to changes in mean values. Analyses of observed temperature and rainfall in many regions of the world have already shown some imperative changes in the extremes. The risks associated with extreme weather events have a great impact on socio-economic activities as well as on the adaptation measures.

A bundle of studies have already taken place about the investigation of climate change and extremes on a very large scale (Easterling et al., 2000; Vincent et al., 2005; Haylock et al., 2006) or at international but few of them made this on a local scale, using local weather stations (Brunetti et al., 2004; Santos and Brito, 2007). The Intergovernmental Panel on Climate Change (IPCC) reports (2001 and 2007) evidenced the need for more detailed information about regional patterns of climate change and the need for the relevancy of future regional climate change. Dufek and Ambrizzi (2008) confirmed that the factors identified through the analysis of recent climate variability can also be important to address the changes in future.

It is widely considered that with the increase of temperature, the water cycling process will be accelerated, which will possibly result in the increase of precipitation amount, intensity and frequency. Global Climate Models (GCMs) indicate the possibility of substantial increases in the frequency and magnitude of extreme daily precipitation (Wang et al.2008). So this is the high time to think about the climate change in regional scale to address the future adaptation measures, especially for the third world country like Bangladesh.

Haor and the wet lands have considered as among the first quickly responded sector for the climate change. Basically the Haor and wet lands of the country has an exclusive hydrological and ecological importance. In Bangladesh Haors are enriched with various aquatic biodiversities along with 140 species of fish. About 8000 migratory wild birds visit the area annually. The extreme flashy character of the rivers and high rain fall compare to other part of the country in the region causes frequent flash floods in the Haor
In Bangladesh, Haors are quite visible in the older floodplains According to the Bangladesh Haor and Wetland Development Board, the total number of the Haor in Bangladesh is near about 414 which are covering an area of approximately 24,500 km². In this study seven Haoer districts (Sunamganj, Habiganj, Maulvibazar, Netrakona, Kishorgonj, Sylhet, Brahmanbaria) are analyzed under 4 BMD stations (Sylhet, Srimongal, Mymensing and Comilla).

This study attempts to provide new information on trends, in regional scale, using long-term records of daily air temperature and rainfall over the Haoe area of Bangladesh, through the analysis of different indices based on observational data from four BMD stations in the region. This analysis can play an important role toward the adaptation strategy for the impact of extreme events of climate change for Hoar area of Bangladesh.

2. METHODOLOGY

For this study, seven Haor districts (Sunamganj, Habiganj, Maulvibazar, Netrakona, Kishorgonj, Sylhet, Brahmanbaria) of Bangladesh have taken account for analysis under preliminary selected five BMD stations (Sylhet, Srimongal, Mymensing, Comilla and Joydebpur). The selection of the BMD stations have made on the basis of the thessen polygon generation by Arc GIS 10.1. All the selected stations have covered near about the whole area of the all Haor districts (Figure: 02), but due to unavailability of sufficient data Joydebpur weather station have been excluded from the study. Daily maximum and minimum surface air temperature and total daily rainfall data from those stations have further analysed by RClimdex 1.0 software for extreme event trend analysis.

2.1 Data inventory and quality control

Daily maximum and minimum surface air temperature data were taken from 4 meteorological stations across the seven Haor districts of Bangladesh as showen in Figure 01, between 23.46- 24.89º N latitude and 90.42- 91.88º W longitude and, in general, for the period between 1960 – 2008 . The station locations are shown in Figure 02; and their names and locations are shown in Table 1. The Center for Environment and Geographical Information Services (CEGIS) provided the data.

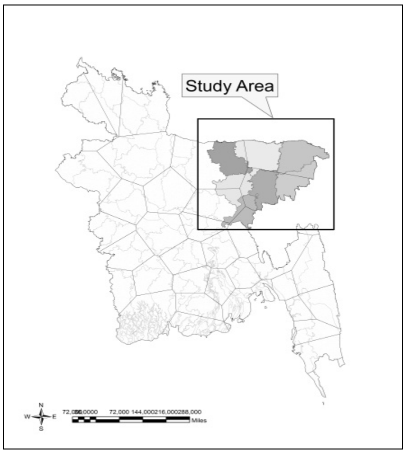

Figure 01: Location of seven Haor districts of Bangladesh

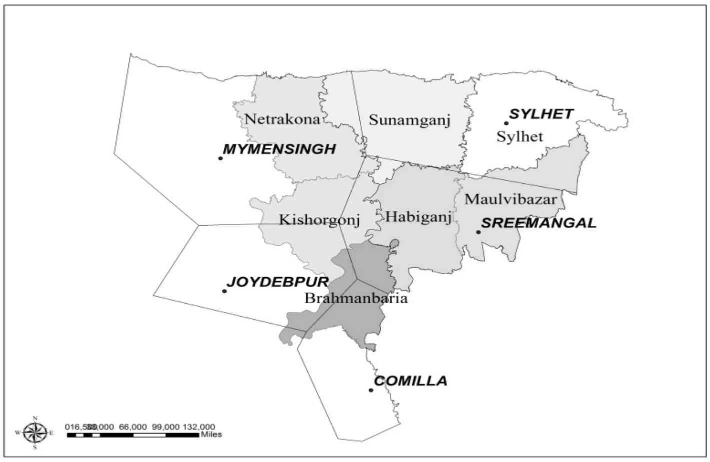

Figure 02: Thessen polygon of selected stations over Haor districts of Bangladesh

Table 01: Selected stations for the study

Station ID	Station Name	Latitude	Longitude	Start	End	Elevation (m)
10609	Mymensingh	24.74000	90.42000	1960	2008	18.44000
10705	Sylhet	24.89000	91.88000	1960	2008	33.94000
10724	Sreemangal	24.30000	91.73000	1960	2008	22.54000
11313	Comilla	23.46000	91.18000	1960	2008	6.54000
41917	Joydebpur	23.98000	90.45000	1960	2008	8.40000

In this study an exhaustive data quality control was applied, because indices of extremes are sensitive to changes in station, exposure, equipment, and observer practice (Haylock*et al*., 2006). Data Quality Control (QC) is a prerequisite for determining climatic indices. The quality control of RClimdex software performs the following procedure: 1) Replaces all missing values (currently coded as -99.9) into an internal format that the software recognizes (i.e. NA, not available), and 2) Replaces all unreasonable values into NA. Those values include daily maximum temperature less than daily minimum temperature. In addition, QC also identifies outliers in daily maximum and minimum temperature. The outliers are daily values outside a region defined by the user. Currently, this region is defined as *n* times standard deviation (*sdt*) of the value for the day, that is, (mean – *n* x *std*, mean + *n* x *std*), where *std*for the day and *n* is an input from the user (Zhang and Yang, 2004; Vincent et al., 2005).

2.2 Methodology and approach

In this study the RClimdex 1.0 software developed by Zhang and Yang (2004) at the Canadian Meteorological Service was used to obtain the climatic extremes indices following methodologies of Zhang et al. (2005) and Haylock et al. (2006). RClimdex provides 27 indices in total (including temperature and precipitation indices). However, only 9 indices based on air temperature data and precipitation data were chosen for this study (Table 2) that better explain the climate behaviour of Haor area of Bangladesh. The resulting series were analyzed through trends. The slopes of the annual trends and their statistical significance to climate indices were calculated based on non-parametric Mann–Kendall test and least square method in order to detect trends within the time series. The Mann–Kendall test has proven to be useful in determining the possible existence of statistically signifi can't trends assuming a 95% probability level (Haylock et al., 2006; Dufek and Ambrizzi, 2008). The products of the test are the statistics S and Z. A positive value of S indicates upward trend and a negative value a downward trend while Z determines the significance or the acceptance or rejection of the null hypothesis, H0, which states that the dataset is formed by n independent and identically distributed random variables. When H0 is rejected at a given significance level, α, one can say that the dataset has a significant trend (Satyamurty et al, 2008). Further details can be obtained in Partal and Kahya (2006).

Table 03 - Definition of extreme air temperature indices used in this study

Indices	Name	Definition	Units
SU	Summer Days	Annual count when TX(daily maximum)>25ºC	Days
TXx	Max Tmax	Monthly maximum value of daily maximum temp	ºC
TNn	Min Tmin	Monthly minimum value of daily minimum temp	ºC
DTR	Diurnal temperature range	Monthly mean difference between TX and TN	ºC

To run the RClimdex 1.0 software the input data file has several requirements: 1) ASCII text file; 2) Columns sequence: Year, Month, Day, Precipitation (PRCP), Maximum air temperature (TMAX), and Minimum air temperature (TMIN). (NOTE: PRCP units = millimeters and TMAX and TMIN units = degrees Celsius); 3) the format as described above was space delimited (e.g. each element was separated by one or more spaces); 4) for data records, missing data were coded as -99.9 (in this study the precipitation values were replaced by -99.9) and data records were in calendar date order (Zhang and Yang, 2004).

Table 03 - Definition of extreme precipitation indices used in this study

Indices	Name	Definition	Units
Rx5day	Max 5-day precipitation amount	Monthly maximum consecutive 5-day precipitation	Mm
R20	Number of very heavy precipitation days	Annual count of days when PRCP>=20mm	Days
CDD	Consecutive dry days	Maximum number of consecutive days with RR<1mm	Days
R99p	Extremely wet days	Annual total PRCP when RR>99th percentile	mm
PRCPTOT	Annual total wet-day precipitation	Annual total PRCP in wet days (RR>=1mm)	mm

3. RESULTS

Table 04 shows the decadal trends of the extreme indices of air temperature and rainfall in Haor districts for four BMD stations. The bold and highlighted values represent significant level of 5% ($p<0.05$), and values only highlighted represent significant level of 10% ($0.05<p<0.1$). From the analysis it have been observed that, the significant change with 5% or 10% level have not seen from any of extreme precipitation indices though some stations have represented some significant changes for the extreme air temperature indices with 5% and 10% of significant level.

Table 04: Trends of extreme indices used in the study

Station ID	Station Name	SU	TXx	TNn	DTR	Rx5day	R20	CDD	R99p	PRCPTOT
10609	MYMENSINGH	-0.005	**-0.043**	-0.044	**-0.038**	0.415	0.112	0.09	1.384	6.888
10705	SYLHET	**0.444**	**0.026**	**0.031**	0.007	0.563	-0.009	0.161	-0.722	-0.059
10724	SREEMANGAL	-0.039	**-0.026**	-0.004	**-0.024**	0.024	0.139	-0.503	-0.643	7.276
11313	COMILLA	NA	NA	-0.007	NA	3.132	0.2	0.568	5.394	8.522

For Mymensingh station, a decreasing decadal trend have observed with 5% significant level for monthly maximum value of daily maximum temperature (TXx) and monthly mean difference between TX and TN (DTR). This analysis have represented that the daily count of maximum value of daily maximum temperature is declining for last 50 years in Netrakona and Kishorgonj districts. The graphical representation of RClimdex is given below in Figure 03.

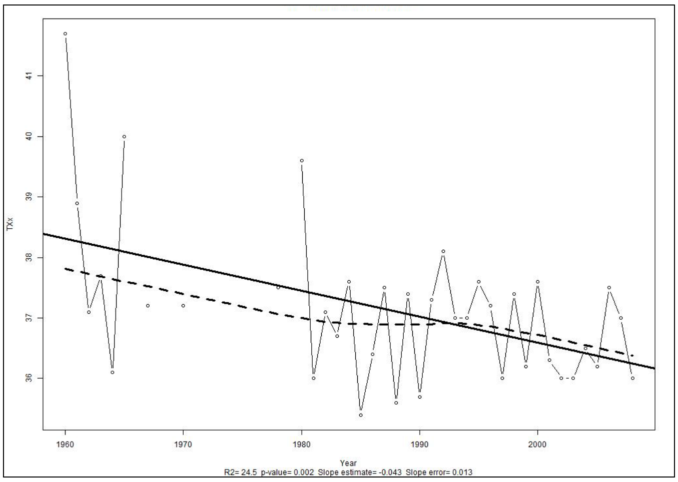

Figure 03: Decadal trend of monthly maximum value of daily maximum temperature (TXx) for Mymensingh

For the Sylhet station, that have completely represented two Haor districts Sylhet and Sunamganj, an increasing trend with steep slope have observed for annual count of summer days when the daily maximum temperature is more than 25ºC (SU25) as shown in Figure 04. It proves that the number of extreme hot days is increasing with a rate of 0.44 ºC per decade in Sylhet region. Concurrently, the monthly minimum value of daily minimum temperature (TNn) have also represented an increasing tendency with a decadal rate of 0.016 ºC (Figure 05) and both change of the historical trend were significant at 5% significant level.

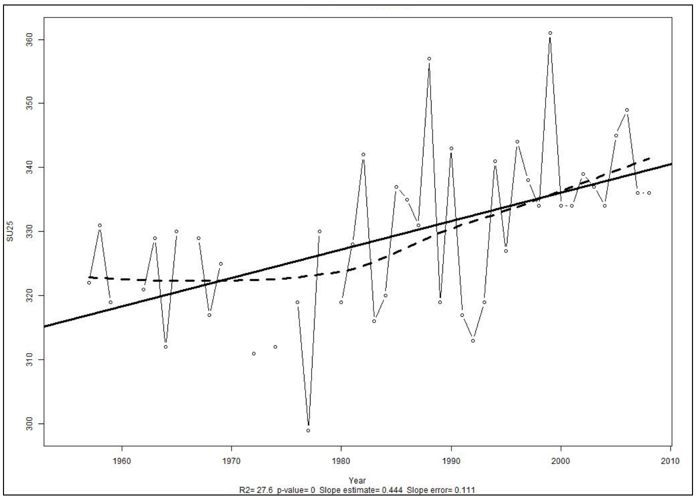

Figure 04: Decadal trend of annual count of summer days when the daily maximum temperature >25ºC (SU25) for Sylhet

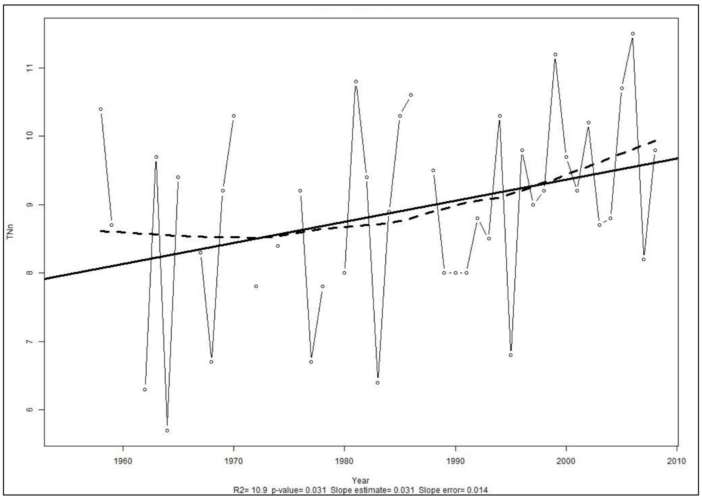

Figure 05: Decadal trend of annual Monthly minimum value of daily minimum temperature (TNn) for Sylhet

Trend of yearly observed monthly mean difference between daily maximum and daily minimum temperature (DTR) have shown in Figure 06 where a decreasing tendency for Srimongal station that have represented two Haor districts Maulvibazar and Habiganj with a rate of 0.024 ºC decrease per decade. The changes for monthly maximum value of daily maximum temperature (TXx) of this station have also represented a decreasing nature with a decadal rate of 0.26 ºC at 5% level of significance.

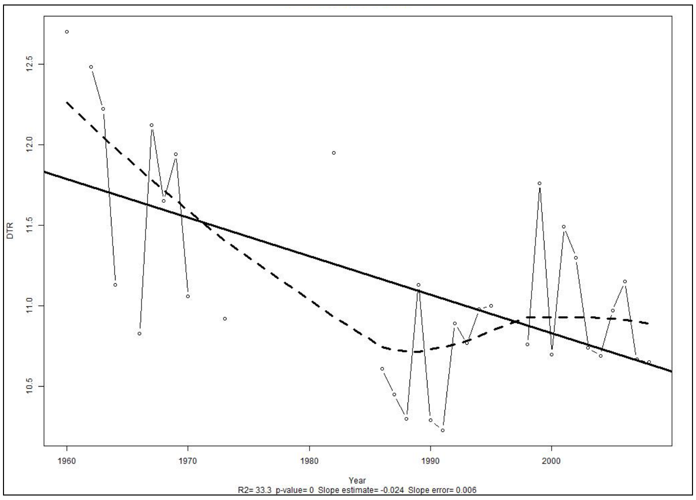

Figure 06: Decadal trend of annual monthly mean difference between daily maximum and daily minimum temperature (DTR) for Srimongal

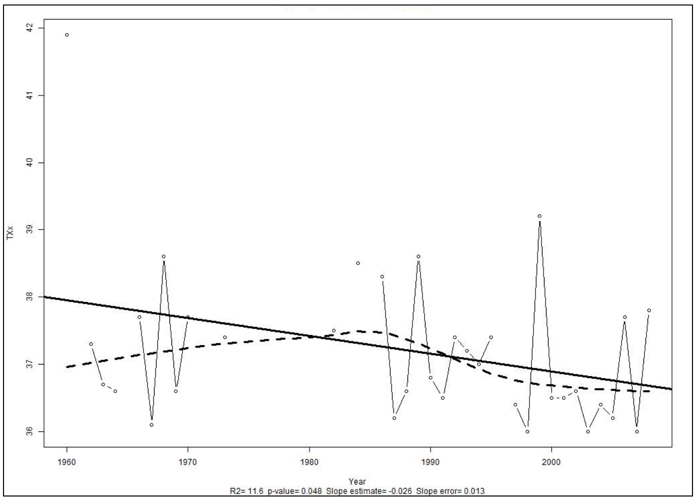

Figure 07: Decadal trend of monthly maximum value of daily maximum temperature (TXx) for Srimongal

For extreme precipitation indices analysis, the highest increasing change have observed for Comilla station for annual total wet-day precipitation when annual total rainfall in wet days is >=1mm (PRCPTOT) at a rate of 8.5mm per decade and for extremely wet days when annual total rainfall is >99th percentile (R99p) at a rate of 5.4mm per decade. A decreasing trend have observed for PRCPTOT, R99p and for number of very heavy precipitation days when annual count of days with >=20mm rainfall (R20) in Sylhet and Sunamganj districts.

4. CONCLUSIONS

The extreme event analyses under this study have represented the trends in nine annual extreme indices of air temperature and rainfall for the Haor districts of the country. The analyses have been carried out for four meteorological BMD stations, in general, for a period between 1960 to 2008, which characterizes a long-term period which represents the occurrence, frequency and changes of the events were significant or not. A significant increase have seen for Sylhet and Sunamganj districts where annual count of summer days with more than 25ºC daily maximum temperature have increased that expanded the summer time in this area. At the same time, the decadal change of the monthly minimum value of daily minimum temperature also represented an increasing tendency that have proved that the number of cold days have increased significantly in this two districts within last 50 years. For Maulvibazar and Habiganj, yearly observed monthly mean difference between daily maximum and daily minimum temperature have represented a decreasing trend that explained that the difference between summer days and winter days have detoeated in this area. This study have also found that the monthly maximum value of daily maximum temperature have also decreased significantly in Maulvibazar and Habiganj districts. Under this study, the precipitation indices analyses have shown no significant changes in any of the weather stations data in Haor area. Decreasing trends have observed for annual total wet-day precipitation and extremely wet days in Sylhet region while the oposit nature has observed for Mymensingh station where both of those indicec have shown increasing trends.

REFERENCES

ALEXANDER, L. V., HOPE, P., COLLINS, D., TREWIN, B., LYNCH, A., NICHOLLS, N. Trends in Australia’s climate means and extremes: a global context. Australian Meteorological Magazine, v. 56, p. 1-18, 2007.

BRUNETTI, M., BUFFONI, L., MANGIANTI, F., MAUGERI, M., NANNI, T. Temperature, precipitation and extreme events during the last century in Italy

DUFEK, A. S., AMBRIZZI, T. Precipitation variability in São Paulo State, Brazil. Theoretical and Applied Climatology, v. 93, p. 167-178, 2008.

DUFEK, A. S., AMBRIZZI, T. Precipitation variability in São Paulo State, Brazil. Theoretical and Applied Climatology, v. 93, p. 167-178, 2008.

EASTERLING, D. R., EVANS, J. L., GROISMAN, P. Y., KARL, T. R., KUNKEL, K. E. AMBENJE, P. Observed variability and trends in extreme climate events. Bulletin of American Meteorological Society, v. 81, p. 417-425, 2000.

HAYLOCK, M. R., PETERSON, T. C., ALVES, L. M., AMBRIZZI, T., ANUNCIAÇÃO, Y. M. T., BAEZ, J., BARROS, V. R., BERLATO, M. A., BIDEGAIN, M., CORONEL, G., GARCIA, V. J., GRIMM, A. M., KAROLY, D., MARENGO, J. A., MARINO, M. B., MONCUNILL, D. F., NECHET, D., QUINTANA, J., REBELLO, E., RUSTICUCCI, M., SANTOS, J. L., TREBEJO, I., VINCENT, L. A. Trends in total and extreme South American rainfall 1960-2000 and links with sea surface temperature. Journal of Climate, v. 19, p. 1490-1512, 2006.

INTERGOVERNMENTAL PANEL ON CLIMATE CHANGE, IPCC. Climate Change 2001 – The Scientific Basis. Contribution of Working Group I to the Third Assessment Report of the IPCC. Cambridge Univ. Press, Cambridge, 2001.

INTERGOVERNMENTAL PANEL ON CLIMATE CHANGE, IPCC. Climate Change 2007 – The Physical Science Basis. Contribution of Working Group I to the Fourth Assessment Report of the IPCC. Cambridge Univ. Press, Cambridge, 2007.

IPCC. Climate Change 2007 – The Physical Science Basis. Contribution of Working Group I to the Fourth Assessment Report of the IPCC. Cambridge Univ. Press, Cambridge, 2007.

PARTAL, T., KAHYA, E. Trend analysis in Turkish precipitation data. Hydrological Processes, v. 20, p. 2011-2026, 2006.

SANTOS, C. A. C., BRITO, J. I. B. Análise dos índices de extremos para o semi-árido do Brasil e suas relações com TSM e IVDN. Revista Brasileira de Meteorologia, v. 22, p. 303-312, 2007.

SATYAMURTY, P., CASTRO, A. A., TOTA, J., GULARTE, L. E. S., MANZI, A. O. Rainfall trends in the Brazilian Amazon Basin in the past eight decades. Theoretical and Applied Climatology, 2008, DOI10.1007/s00704-009-0133-x.

VINCENT, L. A. Trends in total and extreme South American rainfall 1960-2000 and links with sea surface temperature. Journal of Climate, v. 19, p. 1490-1512, 2006

VINCENT, L. A., PETERSON, T. C., BARROS, V. R., MARINO, M. B., RUSTICUCCI, M., CARRASCO, G., RAMIREZ, E., ALVES, L. M., AMBRIZZI, T., BERLATO, M. A., GRIMM, A. M., MARENGO, J. A., MOLION, L., MONCUNILL, D. F., REBELLO, E., ANUNCIAÇÃO, Y. M. T., QUINTANA, J., SANTOS, J. L., BAEZ, J., CORONEL, G., GARCIA, J., TREBEJO, I., BIDEGAIN, M., HAYLOCK, M. R., KAROLY, D. Observed trends in indices of daily temperature extremes in South America 1960–2000. Journal of Climate, v. 18, p. 5011–5023, 2005.

WANG, W., CHEN, X., SHI, P., VAN GELDER, P. H. A. J. M. Detecting changes in extreme precipitation and extreme streamflow in the Dongjiang River Basin in southern China. Hydrological Earth Systems Science, v. 12, p. 207-221, 2008.

ZHANG, X., HEGERL, G., ZWIERS, F. W., KENYON, J. Avoiding inhomogeneity in percentile-based indices of temperature extremes. Journal of Climate, v. 18, p. 1641–1651, 2005.

ZHANG, X., YANG, F. RClimDex (1.0) User Guide. Climate Research Branch Environment Canada. Downsview (Ontario, Canada), 2004, 22p.

ENVIRONMENTAL IMPACT ASSESSMENT ON KALISHURI TO SURJOMONI UNION ROAD CONSTRUCTION PROJECT AT BAUPHAL, PATUAKHALI

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ABSTRACT

Environmental Impact Assessment (EIA) is a decision making process to analyze environmental impacts.This paper is related to EIA of proposed six kilometer road from Kalishuri to Surjomoni Union at Bauphal, Patuakhali that conducting by Local Government and Engineering Department (LGED). The proposed area is so far behind to good communication facilities within the different parts of this region due to absence of road connecting to other area. The significance of the proposed road involves directly to the people of that region in developing their social, economic and environmental condition.The aims of the study are to find out the potential environmental impacts, mitigation measures and to compute the environmental impact value (EIV) of the project.The key findings of this study include environmental positive and negative impacts mainly decreasing plant biodiversity, hydrology and agricultural lands. The study reveals that the EIV is -1 (negative one) in regarding to three environmental factors named ecological, physico-chemical and human interests. The mitigating options of various impacts on the project are environmental friendly technology, conservation of road side plantation, agricultural lands etc.The EIA helps to complete an environment friendly project by providing sound management on environmental impacts and means of preventing or reducing those impacts.

Keywords: Environmental Impact Assessment (EIA), Environmental Impacts, Mitigation Measures, Environmental Impact Value (EIV), Road Construction.

1. INTRODUCTION

Environmental impact assessment is a process which identifies the possible impacts of a project, policies, plans or programmes on the environment prior to the project actually taken place so that redesign of the project can minimize, prevent or compensate impacts properly and accordingly. The purpose of the assessment is to ensure that decision makers consider the ensuing environmental impacts when deciding whether or not to proceed with a project. Environmental Impact Assessment (EIA) report gives us a future scenario of environment which may be changed by the impact of any project. Project construction and operational authority have to be very careful to identify that what are the effects of project to human beings, plants, animals and environment either their project will be fail. Environmental protection issues have to be considering while preparing and planning for constructing a road.The project involves the construction of road from kalishuri to Surjomoni union in Bauphal upazila under Patuakhali district. This project is of road, which is going to be constructing having a length of 6 kilometers:2 (Two) kilometers kacha road is already existing there. But the road is not suitable for public transportation. At time of rainy season it's become very muddy and slippery and it is quite impossible for the local people to transport through it.This is one of the most important proposed roads from kalishuri to Surjomoni union that could be very important for this areas people regarding to communication and also development for two unions enhancing transportation, business opportunities, education, health care system and all parts of social values etc. also its associated areas.

2. METHODOLOGY

To fulfil the objective of the study both primary and secondary data were needed. All the necessary data have been collected from various sources**.**

2.1 Primary data collection

The study was conducted mainly primary data collection sources includes through personal observation, site visiting, open discussion with Road construction authority, group discussion, face to face questionnaire interviews and informal interviews. Lot of photographs was also needed to illustrate the present status and probable impacts on environment as a result of construction of this road of the study area. Most of these photographs have been collected directly from field survey.

Figure 1: Data collection through face to face conversation from Bauphal Upazila office

Secondary data collection

The secondary data for the preparation of the report were collected from Local Government and Engineering Department (LGED), Bangladesh Roads and highway department and from different statistical reports, relevant research papers, books and many national and international journals have also been reviewed for this study. Firstly the probable environmental impacts are identified and then quantifying those impacts on the common base then mitigations measures to the significant impacts are being provided and analysis of the alternatives of the significant impacts and finally documented the EIA report. The region which is covered by the proposed project is far away from numerous development activities like transportation. The population there is depriving from different shorts of benefits due to lack of proper communication and transportation routes.

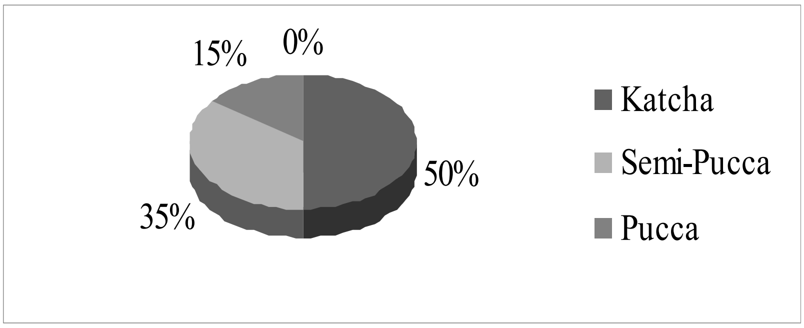

Fig 2: Status of transportation system of the study area

The study reveals that 50% roads of the proposed area are katcha, 35% are semi-pucca and only 15% roads are pucca. For this reason people of that area faces various transporational problem especially in weter season.

2.2 Description of the Project

The project involves the construction of a road near the town of Patuakhali south-western Bangladesh, around 80 kilometres (km) from Barisal. Farming (mainly rice) is the predominant livelihood. Kalishuri itself is a small town of about 15,000 people, which is served by the national highway, upazilla highway and river way to Barisal and also the capital city of Dhaka. The area is among the poorest in Bangladesh. The location of the area is depicted in map 1. The study area situated near the coastal area and the transportation system is not well developed of this area.

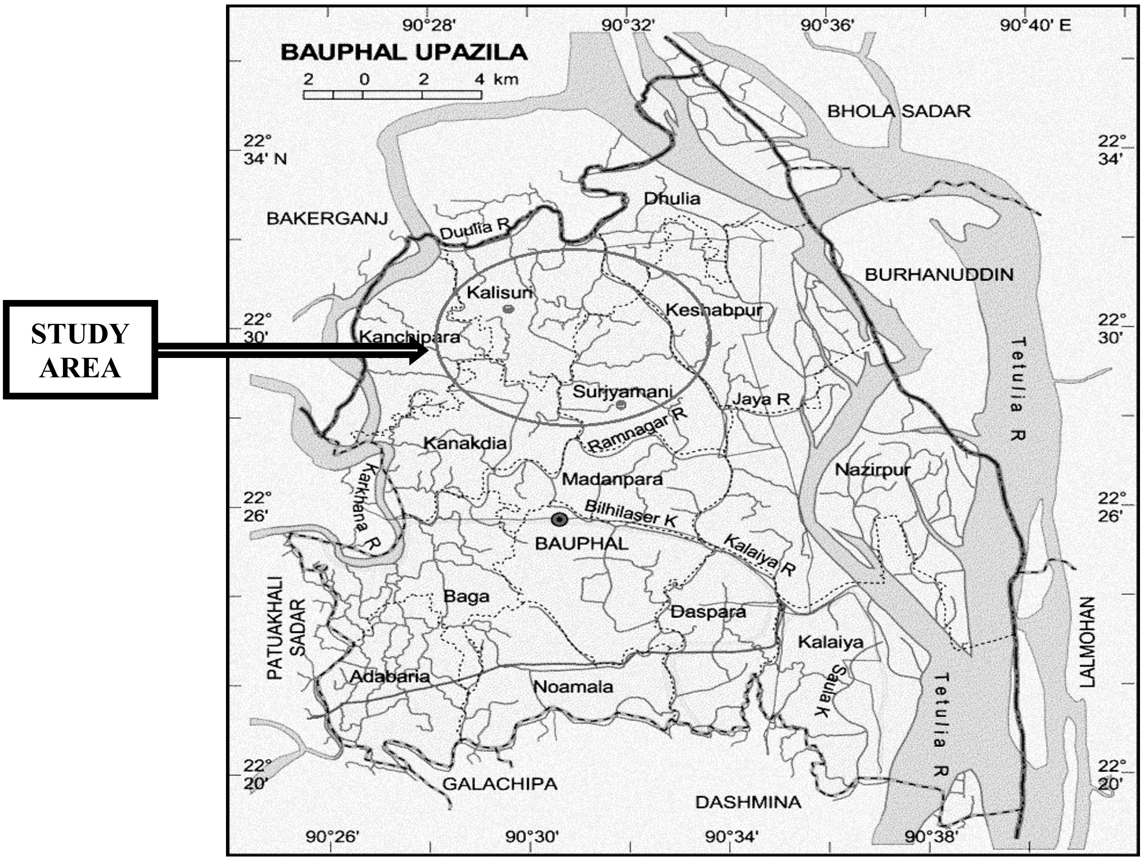

Figure 3: Map of the study area

Table 1: At a glance Kalishuri to Surjomoni road construction project

Sl. No.	Name	Description
01	Project type	Road construction
02	Project Name	Environmental impact assessment on Kalishuri to Surjomoni union Road construction, Bauphal, Patuakhali.
03	Project Location	Kalishuri, Bauphal, Patuakhali
04	Project Duration	2 (Two) years
05	Selected Route	The route was selected from Kalishuri to Surjomoni road
06	Adjacent area	The adjacent area of this road is Kabirkhathi, Ghulbag, and Chhitka.
07	Proposed Length	The length of this road was proposed is about six kilometer.
08	Total Budget	1.5 Corers
09	Authority	The Local Government and Engineering Department (LGED) in Bangladesh take this road construction project

2.3 Description of the Environment

The project area lies within the sub-humid tropical zone and has a mean annual rainfall that ranges from 1100 to 1230mm. it experiences two main alternating seasons: dry and wet seasons. Rainfall lasts from April/May to August/September.Land use within the project area is dominated by agriculture, most agricultural fields are harvested two to three times per year. The main crop is rice. The area is characterized by low salinity concentrations very low arsenic concentrations in groundwater in unconsolidated. Some macro-ecosystem types are found in the project area including Cultivated land, Roadside vegetation, Homestead vegetation, Wetlands areas (rivers, canals, beels, ponds, floodplains). Most houses are study area is tin shed, semi-pucca and woody type. The population of the study area is estimated at 15000 in the year of 2007. The results in an average density of 554 people square per square kilometer. Most of the people are directly involved in agriculture but a minor portion of people doing many works in Dhaka and others city for their livelihood. Educational status of the study area are not satisfactory (only 40%, n=100) due to lack of institutions, lack of awareness, undeveloped transportation etc. Most soils have elevated acidity at the surface, low acidity at depth, and a wide range of water drainage characteristics.

Figure 4: Present condition of proposed route line

3. ILLUSTRATIONS

3.1 Screening of the Potential Environmental Impacts and Mitigation Measures

The project is complex, with various components and subcomponents. These EIA covers all significant impacts on the environment. The more complex issues are discussed in detail, and the mitigation measures are explained and listed. There are three environmental component of the study area which can provide positive as well as negative impact of the project.

Table 2: Environmental parameters of the proposed project

Environmental Parameters of the selected Road construction area (Kalishuri-Surjomoni Union)		
Ecological	Physcio-chemical	Human Interest
Fisheries Forest Plantation Wetland and wetland Habitat Biodiversity Eutrophication	**Land** ☐Erosion and Siltation ☐Backwater Effect ☐Bank stability ☐Drainage ☐Soil characteristics **Surface water** ☐Regional Hydrology ☐Silt Load ☐Water Pollution **Groundwater** ☐Regional Hydrology ☐Recharge ☐Water table ☐Water Pollution **Atmosphere** ☐Air pollution ☐Dust Pollution ☐Noise Pollution	**Health** ☐Diseases ☐Sanitation ☐Nutrients **Aesthetic** ☐Landscape ☐Recreation **Socio-Economic** ☐Land Loss ☐Crop Production ☐Regionalization ☐Irrigation ☐Navigation ☐Flood Control ☐Transport ☐settlement ☐Employment ☐Conflict

The construction phase of the development has the potential to produce the most intrusive impacts, be they from noise, dust, or atmospheric emissions. Although transient in nature, these can cause disturbance to work, leisure and sleep. However, because of the nature of the sources there is usually considerable scope for mitigating the impacts.

3.1.1 Ecological impacts and mitigation measures

❖ **Fisheries**

Action

- Due to construction and construction waste loss of breeding, nursery, and feeding ground of fisheries in floodplains

Impact

- Fisheries species reduces because of water pollution by construction waste
- Fisherman becomes unemployed due to filling the canal for road purpose

Mitigation measures

- Provide adequate opening in roads and sluice gate along routes of fish migration
- Compensate the loss by fish culture

❖ **Eutrophication**

Action

- Leafs can fall to the water bodies while cutting road sides trees and disposing waste into water bodies

Impact

- Decreases in water transparency but increased turbidity and Changes in macrophyte species composition and biomass also change the typical color, smell of the water
- Dissolved oxygen depletion and Increased incidences of fish kills

Mitigation measures

- Safe disposal of construction waste and should be keep Clear the upper layer of the water

❖ **Plantation**

Action

- Create space for tree plantation both sides of the road

Impact

- Soil stability will be increased of this road
- Greater economic benefit will create as a result of plantation

❖ **Wetland and wetland habitat**

Action

- Construction debris, waste can mix with nearby water body of adjacent projected area

Impact

- Decrease the depth of river, canal, beel and other wetland areas through sedimentation
- Eutrophication can be seen as a result of construction work

Figure 5 : Wetland adjacent to the proposed road can be polluted due to construction work

Mitigation measures

- Debris should have to dispose away from the construction site and Construction work should be carried out by considering not damaging the wet land and its habitat

3.1.2 Physico-chemical impacts and mitigation measures

- **Water pollution**

Action

- Construction machinery, construction materials can create the water pollution by disposing waste

Impacts

- Causes water pollution and makes the water unsuitable for beneficial users
- Alteration in the flow of water and possibility of mosquito breeding grounds

Figure 6: Water bodies alongside with the proposed route can be polluted due to construction waste

Mitigation measures

- Create the alteration way to pass waste water
- Construction waste should be disposed remote area from the construction area

- **Erosion and Siltation**

Action

- Cutting trees, Materials ,car movement and fuel can create soil pollution

Impact

- Create land degradation and soil erosion of both sides of the road
- Changes soil texture, make infertile land and can form siltation of canal bed and agriculture lands

Mitigation measures

- Provide adequate opening for discharge of flood and accumulated rain water
- To abstain from cutting trees on both road sides and provide proper slope for surface drainage and vegetation cover

- **Drainage Congestion**

Action

- Construction waste and materials can block passage of water

Impact

- Create water logging due to digging the soil outside of the road to filling the road by soil
- Crop damage and loss of agricultural land and provide ground for mosquito breeding

Mitigation measures

- Provide adequate opening for drainage and facilities for pumping of congested water

- **Waste**

Action

- Waste arising from construction work will typically comprise of excavated material, construction waste.

Impact

- Waste can create health related problem of adjacent people of construction area
- Enviornmental pollution (soil,air,water) can be formed

Mitigation measures

- Construction waste, garbage, domestic waste shall be stored only in the specially designated place on the sites and removed regularly
- The system should be includes as contractual requirements and monitored by the environmental team and audited by the checker

❖ **Noise /Sound pollution**

Action

- Due to construction work (machinery activities)

Figure 7: Construction machine that produce sound pollution

Impact

- Long term effect on human hearing of adjacent construction area and also to construction laborers
- Hampered of daily activities such as educational institutions, mosque, temple etc. along construction site

Mitigation measures

- Noisy works shall be delayed where possible to the late morning
- Recommendation to use silenced equipment where practical and available
- The construction workers should wear personal protective equipment(PPE) during working period

❖ **Air Pollution**

Action

- Due to Construction machinery and construction materials

Figure 8: Air pollution creating activities of road construction purpose

Impact

- Loss of air quality and breathing problem can create to the workers and also people along the road sides

Mitigation measures

- Material should be storage outside of settlement area and also low smoke machinery should be use

❖ **Regional Hydrology**

Action

- Regional hydrology can be threatened by sedimentation and hazardous waste dumping

Impact

- Reduces the fish habitat and can increases water pollution
- Increasing duration, severity and frequency of flood and also decreases the hydrology of that area

Mitigation measures

- Road should be construct perpendicular from the canal
- The passing way of flood water should be keep during rainy season

3.1.3 Impact on human interest

While constructing or planning for any development activities not only nature but also the belonging human being also affected by it. Our proposed project also poses several negative and positive impacts on human interest either directly or indirectly. Those impacts are described below

❖ **Generation of Employment Opportunity**

Action

- Need a large number of people for constructing this road

Impact

- Generate employment opportunity of different poor people
- People easily move to their work place and may open different business window both side of the road

Mitigation measures

- Mitigation measures is not necessary because it comprises positive impact

❖ **Regionalization**

Action

- Communication system will be improved

Impact

- Mutual understanding among the people of different region is increased
- People can introduce with different culture, norms and values

❖ **Agricultural Land**

Action

- Due to Roads on agriculture land and construction waste

Impact

- Loss of agricultural land and also decreases the productivity of the land

Figure 9: Agricultural land that can be affected by construction waste and construction purpose

Mitigation measures

- Practice high yield variety (HYV) crops
- Alternative roads way should be considered if possible

❖ **Transportation**

Action

- Six (6)kilometre roads are going to be constructed for better communication purpose

Impact

- Transportation problem will be lessen also increase development of that area
- Networking system will be improved as well as save travel time

❖ **Settlement**

Action

- Road side Settlement can be damaged as a result of construction road

Impact

- Some road side settlement can be partially or fully destroyed for construction purpose
- People migrate can be occurred due to settlement replaced for constructing this road

Figure 10 : Settlement adjacent to the proposed road can be damaged or replaced due to construction purpose

Mitigation measures

- Alternative direction for the construction of the road should be find out to avoid the destruction of settlement if possible and also construction tools have to keep far from the settlement

❖ **Conflict**

Action

- Political enforcements can be occurred

Impact

- Construction work can be hampered due to conflict
- Reduce co-operation among people and also Several people can be injured by conflicts

Mitigation measures

- Proper authority should be maintained in these problem if raised during construction period

4. ENVIORNMENTAL IMPACT ASSESMENT VALUE (EIV) ESTIMATION

❖ **Method of assessment**

Environmental Impact Value

$$EIV = \sum_{i=1}^{n} (Vi)Wi \qquad (1)$$

Where,

V_i = Relative change of the environmental quality of parameters
W_i= Relative importance or weight or parameter
N = total number of environmental parameters

- **Quantification of environmental impact**

Changes of environmental parameters

- Severe (+5 or -5)
- Higher (+4 or -4)
- Moderate (+3 or -3)
- Low (+2 or -2)
- Very Low (+1 or -1)
- No change (0)

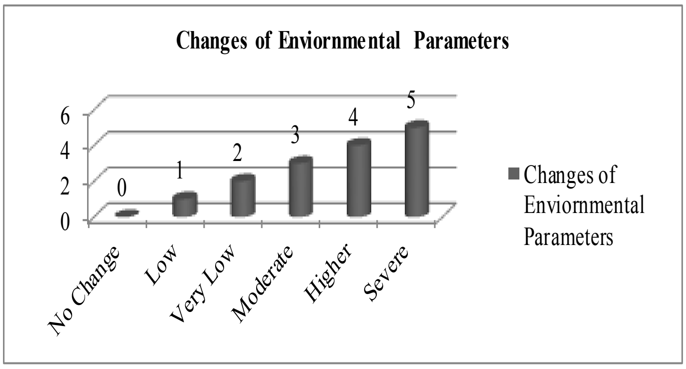

Figure 11: Changes of enviornmental parameters [positive(+) or negative (-)]

Table 3: Estimation of ecological impact value

Parameters	Relative Impact Value (RIV)	Degree of Impact (DoI)	Individual EIV
Fisheries	15	-1	-15
Forest	10	-1	-10
Plantation	25	+1	+25
Wetland and Wetland Habitat	45	0	0
Eutrophication	5	-1	-5
Total			**-5**

Table 4: Estimation of physico-chemical impact value

Parameters	Relative Impact Value (RIV)	Degree of Impact (DoI)	Individual EIV
Erosion and Siltation	9	-1	-9
Water Logging	11	-1	-11
Drainage Congestion	7	-1	-7
Regional Hydrology	20	0	0
Flooding	5	-1	-5
Obstruction ofWaste Flow	12	-1	-12
Dust Pollution	9	-2	-18
Sound Pollution	10	-2	-20
Air Pollution	9	-1	-9
Water pollution	8	0	0
Total			**-91**

Table 5: Estimation of impacts on human interest

Parameters	Relative Impact Value (RIV)	Degree of Impact (DoI)	Individual EIV
Loss of Agricultural land	25	-1	-25
Generation of employment opportunity	10	+4	+40
Transportation	45	+2	+90
Regionalization	5	+2	+10
Conflict	5	-2	-10
Settlement	10	-1	-10
Total			**+95**

Table 6: Total environmental impact estimation

Sector	Total	**Result** (EIV) = $\sum^{n}_{i=1} (V_i) W_i$
Ecological	-5	-1
Physcio-chemical Impact	-91	
Human Interest	+95	

After observing and estimating all the parameters we are able to identify the Environmental Impact Value (EIV) of our project.The Environmental Impact Value (EIV) considering both positive and negative is -1 (negative one). On the basis of this result we can say that this project has very low neagtive impact on the enviornment but if we able to take precautionary measures to eliminate the negative impacts of this project then this project will be succeeded and people get benefits of the project. After considering the all issues we recommend the Kalishuri to Surjomoni Union Road should be constructed. Strict implementation of Enviornmental Management Plan (EMP) brings the best outcome and success of the project in term of reducing negative enviornmental impacts of this project. A proper enviornmental management plan can ensure the enviornmental sustainability of a project. If EMP will implement properly then the negative impacts on enviornment can be minimized but without EMP the negative impacts on enviornment of this project will be much more severe that are depicted in the following graphical presentation.

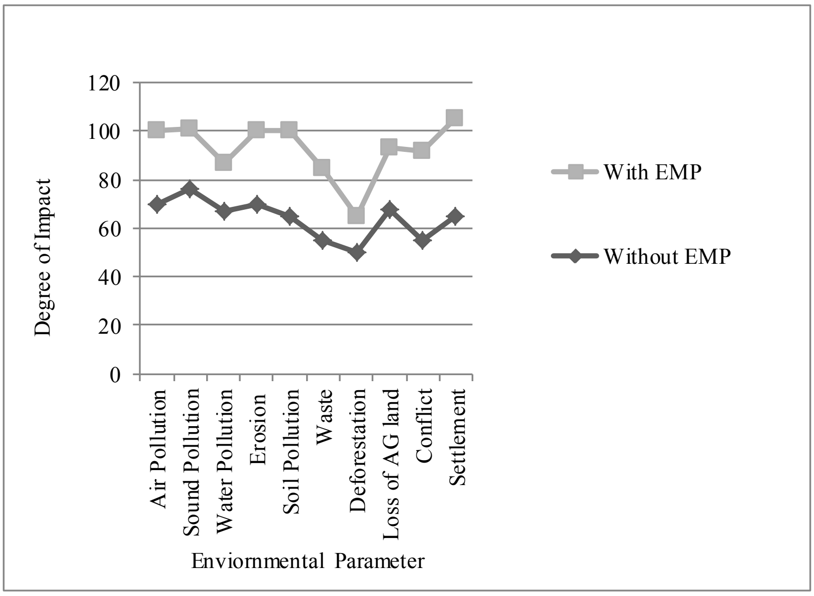

Figure 12: Significance of enviornmentall management plan regarding to with and without EMP

5. CONCLUSION

The project is all about Environmental Impact Assessment (EIA) of constructing from the route Kalishuri to Surjomoni union. Different environmental elements will be degraded by this construction work like wetland, forest, hydrology, fisheries, air, water, soil, agricultural land etc. But those impacts can be minimized by taking precautionary measures and adopting eco-friendly technologies. Also, the mitigation measures would be sustainable and long term durable of the proposed road construction project. Our calculated EIV also shows a very low change on the environment due to this project because it is -1(negative one). Negative impacts can be reduced by proper implementation of enhancing environmental management plan. Except these negative impacts this project is very much suitable for this area in context of transportation, plantation, employment opportunity, regionalization etc. Those Positive impacts strengthen the implementation of the project. The significance of this project in this area is higher than its negative consequence on the environment. EIA helps to the successfully completing of this project. As a result, people benefited as solving transportation problem of the proposed area. If we conscious enough about the environment while constructing this road and follow all guidelines for environmental protection then the project will be a great milestone for the development of this area.

ACKNOWLEDGEMENTS

We would like to express our appreciation and cordial thanks to all the people whose effort and input made it possible for this study to carry on to completion. We sincerely acknowledge the cooperation and help of the all staffs of Bauphal Upazila Parishad and Kalishuri, Surjomoni Union Parishad and LGED in Patuakhali District. We express our deep gratitude to the community's people of the study area for their heartiest cooperation throughout the study period.

REFERENCES

Asia Energy Corporation (Bangladesh) Pty Ltd for the Asian Development Bank (ADB). Environmental Assessment Report, Bangladesh: Phulbari Coal Project.

Feasibility study of Kalishuri to Surjomoni Union Road construction Project, Bauphal, Patuakhali(2013).

George C. Testing for sustainable development through environmental assessment. Environmental Impact Assessment Review 1999;19:175–200.

Goodland, Robert and Robert Tillman. (1996). "Strategic Environmental Assessment", in J.R. Mercier and S. Muntemba (eds.), Environmental Assessment in Africa: A World Bank Commitment. Proceedings of the Durban World Bunk Workshop. Washington, DC: World Bank.

Modi Ashwini Jajda, Shinkar N.P. (2012). Environmental Impact Assessment of Road from Ujjain to Jaora. International Journal of Engineering and Advanced Technology (IJEAT) ,ISSN: 2249 – 8958, Vol.1, Issue-4

Journal of the Eastern Asia Society for Transportation Studies, Vol. 6, pp. 1313 - 1328, 2005.

Rabinovitch, J. (1992). Curitiba, Brazil: Environmental Management Case Study. Urban Management Program. Washington, DC: World Bank.

Technical guidelines Environmental Impact Assessment in Road sector (2011), Public Enterprise "Roads of Serbia" Belgrade.

United Nations Environment Program. (1995). Environmental impact Assessment "Good Practice Guide". Nairobi, Kenya: UNEP.

Proceedings of the 2nd International Conference on Civil Engineering for Sustainable Development (ICCESD-2014), 14~16 February 2014, KUET, Khulna, Bangladesh, ISBN: 978-984-33-6373-2 (CD-Rom)

STUDY ON ENVIRONMENTAL MANAGEMENT PLAN (EMP) OF PROPOSED LEATHER INDUSTRY IN LEBUKHALI, PATUAKHALI

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ABSTRACT

This study represents the assessment and calculation of environmental impacts with an environmental management plan (EMP). It was conducted from 11th July to 30th October. Industrialization has been rapid in Bangladesh during the last decade, particularly in the sectors of leather, pharmaceuticals and industrial chemicals. It is necessary to establish industry for increasing industrial development in Barisal. The aims of the study are to identify of environmental impacts, to suggest some mitigation measures and an EMP. The project involves the establishment and development of a leather industry in Lebukhali near the town of Patuakhali in southern part of Bangladesh. Lebukhali is the transition zone between Barisal and Patuakhali. The geographical coordinates of this area is 22° 27' 0" North, 90° 20' 0" East. An environmental impact assessment (EIA) is designed to enable the environmental effects of a project with economic costs and benefits. The major environmental impact would be air pollution, water pollution and waste siltation. As the human interest is positive, some measures can be recommended to establish the industry. To ensure proper environmental management plan enhance tree plantation programs, dredging of river, waste management and use environmental friendly technology. It is expected that the industry will increase employment opportunity and livelihood security.

Keywords: *Environmental Management Plan (EMP), environmental impacts, mitigation measures, leather industry, industrialization.*

1. INTRODUCTION

The leather industry sector is the fourth largest foreign exchange earner of the country, contributing about 6% of total export earnings. There are at present 214 tanneries in Bangladesh. Among them 200 in the city of Dhaka and 14 others scattered all over the country. In spite of Barisal is a division, there is no industry here. It is necessary to establish an industry in Barisal. Lebukhali is away from the main city area. So we can select it for leather industry. An Environmental Management Plan (EMP) is a detailed plan and schedule of measures necessary to minimize, mitigate, etc. any potential environmental impacts. The EMPs will be maintained as controlled documents to ensure that all relevant parties are informed of any changes in mitigation measures and procedures that could affect the environment. An EIA is also important to ensure the safety of both the workers and the public. The main objectives of the project are:

- Identification of all major environmental impacts on the proposed leather industry.
- To assess the environmental cost-benefit analysis of projects.
- To calculate environmental impact value (EIV).

2. METHODOLOGY

We have survey on the livelihood pattern of Labukhali area. There are two sources of data and relevant information, these are:

- Primary source of data by
  - ✓ Personal observation
  - ✓ Field visit
  - ✓ Public opinion
- Secondary source of data by
  - ✓ Analysis of other relevant industry

2.1 Project Description

The Project involves the establishment and development of a leather industry in Lebukhali near the town of Patuakhali in southern part of Bangladesh. Farming and fishing is the predominant livelihood here. Lebukhali itself is a small union, which is served by the junction of Patuakhali – Dhaka highway. The overall task of this Project is divided into three main components:

- Lebukhali leather industry.
- Environmental Management Plan of this proposed industry.
- Calculating the Environmental Impact Value and giving recommendation on behalf the leather industry in this area.

2.2 Study Area

Original name: Lebukhali.
Geographical location: Patuakhali district, Barisal Division, Bangladesh.
Geographical coordinates: 22° 27' 0" North, 90° 20' 0" East.

Figure 1: Selected area for leather industry

Lebukhali is situated beside Paira River. Most of the people are illiterate and unemployed. Here also live cyclone Sidr affected refugee people. If a tannery industry is established here, many people will get job opportunity. It will play an important role in economy. But it will create negative impact on the environment.

2.3 Description of the Environment

Environmental management plan is designed on the basis of current environment. If an industry is constructed then it will have an adverse effect on environment. And the environment might change. The present environmental situation can be described in 3 categories:

- Meteorology and Climate
- Physical and Biological Environment
- Socio-cultural Environment

2.3.1 Meteorology and Climate

The climate (precipitation), the regular annual variation (monsoon), and other meteorological conditions significantly affect the environmental impact of the Project. For instance, there occurs tidal flooding in the rainy season and hence submerges the river side.

Three seasons are generally recognized: a hot, muggy summer from March to June; a hot, humid and rainy monsoon season from June to November; and a warm-hot, dry winter from December to February.

2.3.2 Physical and Biological Environment

A portion of Lebukhali union is located in the Project area. The Project area is alongside the river and the transition zone between Barisal and patuakhali. Key biophysical features of the environment are:

- The Project area is along the river side, where scattered villages of climate refugee are situated.
- Land use within the Project area is dominated by agriculture.
- Most plots are harvested two to three times per year (average 2.2); the main crop is rice.
- Flooding is common in tidal period.

2.3.3 Socio-cultural Environment

- Most of the people are illiterate. The children are not interest to go to school. They mainly help their parents in household work. Their parents are not aware of childrens education.
- 85% people are muslim and 15% are hindu. The number of educational institution is not enough.
- The main livelihood of the people is fishing and farming. The main crop is rice. People catch fish from the Lebukhali River. Most of the people live below the poverty line.

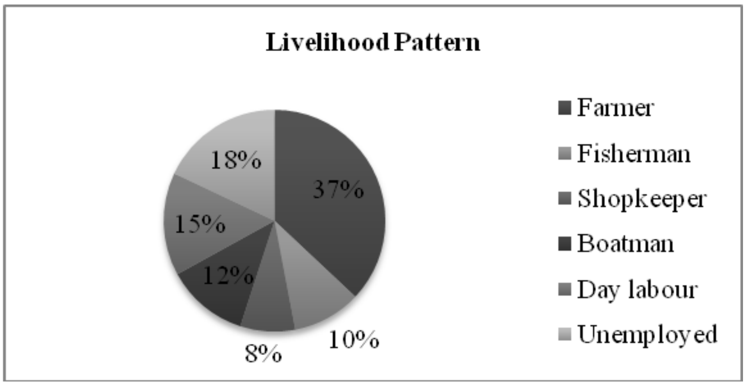

Figure 2: Livelihood pattern of the study area

3. ILLUSTRATIONS

3.1 Risk Assessment

A risk assessment is included in all project components. The risk assessment must consider not only the consequence of a potential residue spill or other accident, but also how often potential accidents are likely to occur, what mitigation measures are put in place to minimize the risk of such accidents occurring, what counter measures are put in place to minimize losses if such accidents occur, and the ability to effectively use counter measures. Undoubtedly the first issue to come up in residue pollution risk analysis of the area is the potential severity of an accident. Severity is determined by a number of factors. Risk is determined by combining the severity with the likelihood of pollution. Once the severity and likelihood are established, then the risk can be determined. Risk may range from extremely high (unacceptable) to low (acceptable) (Gupta Shivam 2007). In general terms, waste products may occur as waste water, solid material, volatile compounds or gasses that are discharged into the air.

3.1.1 Wastewater

An important environmental impact of the animal processing industry results from the discharge of wastewater. Most processes in slaughterhouses, tanneries and dairy plants require the use of water. This water and water used for general cleaning purposes will produce wastewater. The strength and composition of pollutants in the wastewater evidently depend on the nature of the processes involved. Discharge of wastewater to surface waters affects the water quality in three ways: (Mainuddin Khandaker 2003)

- The discharge of biodegradable organic compounds (BOC's) may cause a strong reduction of the amount of dissolved oxygen, which in turn may lead to reduced levels of activity or even death of aquatic life

- Macro-nutrients (N, P) may cause eutrophication of the receiving water bodies. Excessive algae growth and subsequent dying off and mineralisation of these algae, may lead to the death of aquatic life because of oxygen depletion.
- Agro-industrial effluents may contain compounds that are directly toxic to aquatic life (e.g. tannins and chromium in tannery effluents; un-ionized ammonia).

3.1.2 Solid waste

By-products that are not used in any way will be referred to as solid waste. They must be dumped**.**The following types of solid waste may be distinguished:

- Providing livelihood security of the people of Lebukhali and its surrounding area.
- toxic compounds: These compounds require special attention, e.g. special dumping grounds.
- organic compounds: These compounds may require attention under certain conditions because of hygienic reasons or because during decomposition ill odour or leaching problems may arise.
- non degradable compounds: These may be dumped at regular dumping grounds.

3.1.3 Air pollution

Air pollution may cause problems of various :

- global warming, as a result of emissions of CO2;
- changes in the ozone-layer, as a result of emissions of NOx, CH4, N2O and CFC's;
- acid rain, as a result of emissions of SO2 and NH3 health conditions;
- dust and/or bad odour, as a result of emissions of VOC;

3.2 Possible Impact Analysis

Impacts on environment are inevitable by-products of the leather manufacturing process and cause significant pollution unless treated in some way prior to discharge. These industries affect our environment in various ways. The environment is under increasing pressure from solid and liquid wastes emanating from the leather industry. In this project possible environmental impacts are analyzed with respective recommendations. It provides a means to identify the most environmentally suitable option at an early stage, the best practicable environmental option, and alternatives to the proposed initiative; and thus avoid or minimize potentially damaging and costly negative impacts, and maximize positive impacts.(Cipriani 2002)

Table 1: Environmental impact assessment

Ecological	Physico-chemical	Human Interest
Agriculture Fisheries Forest Plantation Wetland Biodiversity	Waste siltation Water pollution Air pollution Drainage congestion	Loss of land Land ownership pattern Commercial and services facilities Generation of employment opportunities Industrialization

3.3 Mitigation and Impact Management

Mitigation is done to avoid, minimize or offset predicted adverse impacts and, where appropriate, to incorporate these into an environmental management plan or system.Strengthening the positive impacts should be necessary to get all its potentiality.

Environmental Impact Value:

$$EIV = \sum_{i=1}^{n} (Vi)Wi \tag{1}$$

Where,

V_i = Relative change of the environmental quality of parameters
W_i= Relative importance or weight or parameter
N = Total number of environmental parameters

The Environmental Impact Value (EIV) considering both positive and negative is -4 (negative four).

For each potential adverse impact the plan for its mitigation at each stage of the project should be documented and costed, as this is very important in the selection of the preferred alternative.
The objectives of mitigation therefore are to:

- Find better alternatives and ways of doing things;
- Enhance the environmental and social benefits of a project
- Avoid, minimise or remedy adverse impacts; and
- Ensure that residual adverse impacts are kept within acceptable levels
- Development of production process, applying clean technology or less polluting technology.
- Improvement of control and supervision of the different production steps in order to ensure proper operation of the different equipment andavoid the discharge of pollutants to the environment

Table 2: Impact assessment of parameters with mitigation measures

Types	Parameters	Action	Impact	Mitigation
Ecological	Loss of fish breeding	Destruction of fish production	• Loss of economy • Loss of fisherman	• Create alternative breeding environment
	Cutting trees	Deforestation	• Reduction in forest cover • Reduction in forest production • Destruction in ecology	• Enhancing tree plantation programmers • Road side tree plantation
	Drying wetland	Drying up of agricultural purpose and destruction of habitat for fish, bird etc	• Reduction of fishery • Reduction of species of fish, bird etc • Destruction of wetland ecology	• Dredging river
Physico-chemical	Waste siltation	Reduce deepness of river	• Siltation of canal bed and agricultural land • Increase turbidity of water	• Dredging river • Proper waste management
	Water pollution	Destruct water quality	• Loss of fish, amphibians and other habitat • Destroy water ecology • Make water unsuitable for user	• Purification of water before using • waste management
	Air pollution	Emission of GHG	• increase air born diseases	• using environmental friendly technology
	Drainage congestion	Blockage of sewer system	• flooding • environmental pollution	• widening drains and sewer channels
Human interest	Loss of land	Loss of agricultural production	• deprives a group of farmers • increase landless • reduce unemployment	• proper land use planning
	Land ownership pattern	Changes in land tenure	• increase conflict • loss of land	• land zoning
	Commercial and services facilities	Industrialization	• generation of employment opportunities • loss of biodiversity	• using green technology

4. ENVIRONMENTAL MANAGEMENT PLAN

The EMP will identify the project management structure and clearly identify the roles and responsibilities with regard to managing and reporting on the construction phase environmental aspects. An Environmental Risk Assessment will be undertaken when developing the EMP. The risk assessment dentifies all aspects of construction that could have an environmental impact and assesses the potential risk and impact of that activity on the environment. Management controls are then devised to eliminate and/or minimise those identified impacts. The assessment would address the potential impacts created during the temporary construction period and any permanent impacts that are influenced by construction methods. Specific environmental issues would be addressed in the EMP and strategic details on how these would be controlled across the project would be provided. A list of potential issues that will need to be addressed in the plan are provided below based on information provided in the Environmental Statement.(Harbour 2007)

- Construction noise and vibration management
- Air quality including dust management
- Sustainable waste management
- Archaeology and heritage management
- Water management (surface and groundwater)
- Management and protection of ecological resources (particularly relating to timing of certain works)
- Contaminated land management
- Resettlement

The EMP would set out objectives and targets for the project that are realistic and relevant for maintaining or improving environmental performance. A programme of monitoring, reporting and auditing of compliance in accordance with any obligations of the planning consent, licences and approvals should also be contained in the EMP to ensure that identified and appropriate control measures are effective.

4.1 Environmental Aspect Identification:

EAs should be identified comprehensively and reviewed regularly to ensure that they are still appropriate (e.g. planned or new developments, or new or modified activities, products and services. EAs can be positive (e.g. waste recycling) or negative (e.g. toxic waste generated). The following are typical EAs to be considered:

- Air emissions
- Water discharges
- Solid and hazardous wastes
- Contamination of land
- Raw material and resource use (water, hides, skin of domestic animal, etc.)
- Local issues (e.g. noise, odour, dust, etc.)
- Hazardous material storage and handling
- Habitat disturbance

Table 3: Considering environmental aspects

Requirements	Technical	Environmental	Level
Legislation Permits Customers Complaints	Process Storage Transfer Transport Utilities Product	Air Water Waste and by-products Land and groundwater contamination Use of raw materials Energy emitted, e.g. heat, radiation, vibration External safety Product	Site / plant Installation / equipment Subcontractor / supplier

4.2 Environmental Policy

Environmental policy refers to the commitment of an organization to the laws, regulations, and other policy mechanisms concerning environmental issuesand sustainability. These issues generally include air and water pollution, solid waste management, biodiversity, ecosystem management, maintenance of biodiversity, the protection of natural resources, wildlife and endangered species. Policies concerning energy or regulation of toxic substances including pesticides and many types of industrial waste are part of the topic of environmental policy. This policy can be deliberately taken to direct and oversee human activities and thereby prevent harmful effects on the biophysical environment and natural resources, as well as to make sure that changes in the environment do not have harmful effects on humans. (Harstad 2012)

- Ensuring continuous observance of applicable laws and regulations
- Designing for the environment by developing and implementing new technology to deliver more environmentally friendly solutions
- Allocating responsibilities for environmental management in all company activities, ensuring that all employees are aware of their individual responsibility in upholding this policy.

4.3 EMP Preparation

Environmental management plan (EMP) consists of the set of mitigation, monitoring, and institutional measures to be taken during implementation and operation to eliminate adverse environmental and social impacts, offset them, or reduce them to acceptable levels. The plan also includes the actions needed to implement these measures. More specifically, the EMP includes the following components.

4.3.1 Mitigation

The EMP identifies feasible and cost-effective measures that may reduce potentially significant adverse environmental impacts to acceptable levels. The plan includes compensatory measures if mitigation measures are not feasible, cost-effective, or sufficient. (a) Identifies and summarizes all anticipated significant adverse environmental impacts (b) Describes each mitigation measure with technical details, including the type of impact to which it relates and the conditions under which it is required together with designs, equipment descriptions, and operating procedures, as appropriate.(c) Estimates any potential environmental impacts of these measures; and (d) Provides linkage with any other mitigation plans required for the project.

4.3.2 Monitoring

Environmental monitoring during project implementation provides information about key environmental aspects of the project, particularly the environmental impacts of the project and the effectiveness of mitigation measures. Specifically, the monitoring section of the EMP provides or sufficient. Identifies and summarizes (a) a specific description, and technical details, of monitoring measures, including the parameters to be measured, methods to be used, sampling locations, frequency of measurements, detection limits (where appropriate), and definition of thresholds that will signal the need for corrective actions; and (b) monitoring and reporting procedures to (i) ensure early detection of conditions that necessitate particular mitigation measures, and (ii) furnish information on the progress and results of mitigation.

4.3.3 Capacity Development and Training

Specifically, the EMP provides a specific description of institutional arrangements that is responsible for carrying out the mitigatory and monitoring measures (e.g., for operation, supervision, enforcement, monitoring of implementation, remedial action, financing, reporting, and staff training). To strengthen environmental management capability in the agencies responsible for implementation, most EMPs cover one or more of the following additional topics: (a) technical assistance programs, (b) procurement of equipment and supplies, and (c) organizational changes.

4.3.4 Integration of EMP with Project

Individual mitigation and monitoring measures and its assignment of institutional responsibilities, and it must be integrated into the project's overall planning, design, budget, and implementation. Such integration is achieved by establishing the EMP within the project so that the plan will receive funding and supervision along with the other components.

4.4 EMP Structure

Table 4: EMP structure

Task	Environmental Policy	Duration
Baseline assessment	Initial Environmental Review checklist Gap analysis reporting	2 weeks
Environmental Aspect (EA) identification	Environmental Aspect Register	2 weeks – 1 month

Identification & compliance with legal and other requirements	Legal and other requirements register	2 weeks
Evaluating environmental aspects	Environmental aspect identification and evaluation procedure	2 weeks – 1 month
Developing Objectives & Targets with Programmes	List of objectives, targets and programmes	2 weeks
Developing EMS documentation	EMS Manual EMS procedures	1 month
Developing operational control procedures	Operational control procedures and work instructions	1-2 months
Implementation of the EMS	Organisation chart & responsibilities Training plan Training materials Guidance notes for supplier control Communication records Forms for implementing procedures	2-3 months
Checking, audit	Monitoring plan Audit plan Audit checklist Audit report Corrective action & preventive action report	1 month
Review	Management review report	2 weeks

4.5 Environmental Management System

An Environmental Management System (EMS) is a continual business cycle of planning, implementing, reviewing and improving the processes and actions that your company undertakes to meet its environmental obligations and continually improve its environmental performance. An effective EMS is developed on "Plan, Do, Check, Act" (PDCA) model which embodies the concept of continual improvement.(Shahrukh Rafi Khan 1999)

Table 5: Environmental management system

PDCA Cycle	Standards
Plan	Planning Environmental Aspects Legal and Other Requirements Objectives, Targets and Programme(s)
Do	Implementation and Operation Resources, Roles, Responsibility and Authority Competence, Training and Awareness Communication Documentation Control of Documents Operational Control Emergency Preparedness and Response
Check	Checking Monitoring and Measurement Evaluation of Compliance Nonconformity, Corrective Action and Preventive Action Control of Records Internal Audit
Act	Management Review

4.6 Possible Significance

EMP provides the remedial strategies increasingly include the on-site containment of certain types of contamination rather than its full scale removal. Removal of the contamination is likely to cause greater harm to the environment than leaving it undisturbed. ‘an environmental management plan contaminated means a plan which addresses the integration of environmental mitigation and monitoring measures for soil and groundwater throughout an existing or proposed land use. There is some negative impact behind all industry. But industrialization is the pre condition of development. So if we take above environmental management plan, the negative consequences will minimize.

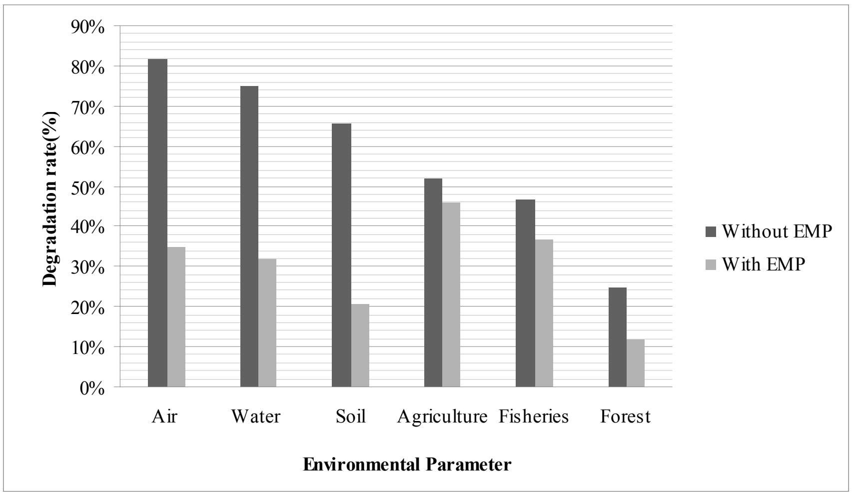

Figure 3: Significance of EMP

5. ANALYSIS OF ALTERNATIVES

Leather industry is an old manufacturing sub-sector in Bangladesh with a long heritage of over six decades. For the economic development of Barisal division it is necessary to establish this industry in Patuakhali district. And Lebukhali is an appropriate location for it. The Project will provide following opportunity. Such as:

- Making available and cheap leather products in southern part.
- Providing direct and indirect employment opportunities.
- Providing livelihood security of the people of Lebukhali and its surrounding area.
- Contributing to the national economy.
- Industrialization of Barisal division.
- Supplying high-quality, low-cost leather product for the domestic market.

5.1 River Transportation Route Alternatives

- River transportation route is very important for carrying different materials use in shipyards. But its alternatives are highway and road transportation.
- But highway destroys the higyway roads. So river transportation is right way to transportation.
- This project is near the river side. So river transportation is much important than other transportation.
- Beside it the road condition is not well.

5.2 Possible Location Alternatives

- Alternative locations are Patuakhali, Bakerganj, Dopdopia or other river port areas, but some problems related to these location.
- The biggest river port situated near the project area.
- There are some industry such as cement factory, chemical factory are situated in Barisal region
- Other location are not appropriate like the project area.
- So under the above consideration Lebukhali is better selection.

5.3 Transportation and Navigation Facilities

- These area is situated alongside the paira river. Different kinds of launches, cargoes, ships and burges are available. Essential raw materials for leather industry can be transport easily.
- Lebukhali is the transition between Dhaka-Patuakhali. But the condition of roads is very bad. If we can repair the road, it will be fruitful for our industry.

5.4 Public Opinion

In spite of having negative impacts, maximum people are interested for industry. Some are not interested by considering negative consequences but these are little in number.

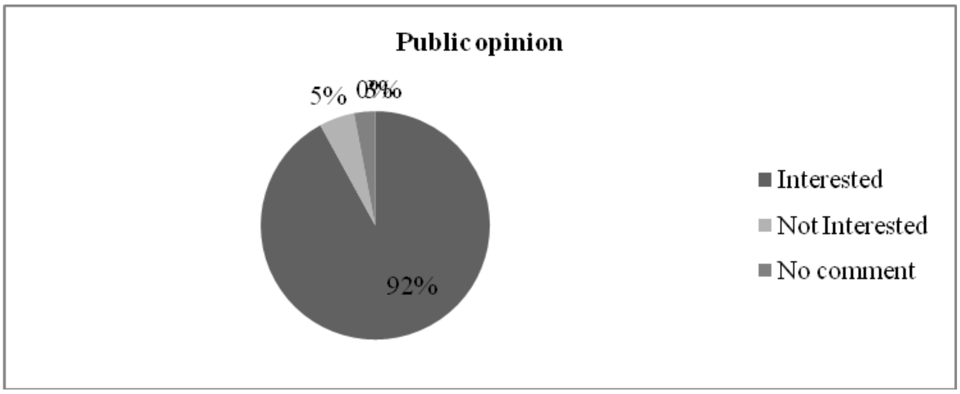

Figure 4: Public opinion

6. CONCLUSIONS

Due to this industry the people of the area can change their livelihood pattern. They can lead a standard life than present. As well as the area will develop. This study has revealed that the tanning activities involve serious environmental hazards. Among them air pollution, water pollution and waste siltation are most significant. By enhancing tree plantation programmers, dredging river, proper waste management and using environmental friendly technology we will able to ensure this industry development as sustainable. Finally, direct recycling of chrome tanning float, recycling of chrome after precipitation, should be taken in tannery industrial activities with a view to ensuring safe, sound and healthy environment for the greater benefit of Bangladesh.

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REFERENCES

A. M. Hossain ―Heavy Metal Concentration in Tannery Solid Wastes Used as Poultry Feed and The Ecotoxicological Consequences, Bangladesh Journal of Scientific and Industrial Research vol. 42, no. 2 pp.397–416, 2007

Bhuiyan, et al. ―Investigation of the Possible Sources of Heavy Metal Contamination in Lagoon and Canal Water in the Tannery Industrial Area in Dhaka, Bangladesh. Environmental Monitoring and Assessment, 2010.

Cipriani, S. (2002). "STUDY OF THE QUALITY CONTROL AND MANAGEMENT ON THE AFRICAN LEATHER SECTOR."

Gupta Shivam , R. G., Ronak Tamra (2007). "CHALLENGES FACED BY LEATHER INDUSTRY IN KANPUR."

Harbour, H. (2007). "Construction environmental management plan (CEMP)."

Harstad, B. (2012). "Bye Coal! A Case for Supply-side Environmental Policy." The University of Chicago Press.

Huq, S. M., ―Critical Environmental Issues Relating to Tanning Industries In Bangladesh. In: Naidu et al. (eds), Towards Better Management of Soils Contaminated with Tannery waste, ‖ in Proceedings of a workshop held at the Tamil Nadu Agricultural University, Coimbatore, India, 31 January to 4 February, 1998.

Mainuddin Khandaker, D. M. I. S. (2003). "Country Case Study on Environmental Requirements for Leather and Footwear Export from Bangladesh."

M. G. Rusal, I. Faisal, K. M. M. Kamal, ―Environmental pollution generated from process industries in Bangladesh, ‖ Journal of Environmental Pollution. vol. 28, no. 2, pp. 44-161, 2006.

Shahrukh Rafi Khan, M. A. K., Abdul Matin Khan, Haider Ghani and Sajid Kazmi (1999). "Environmental Impacts and Mitigation Costs Associated with Cloth and Leather Exports from Pakistan."

UNIDO, United Nations Industrial Development Organization, Cost of Tanned Waste Treatment, 15th Session of the Leather and Leather Products Industry Panel, Leon, Mexicom, 2005.

GENERATION OF AMMONIA IN DELIMING OPERATION FROM TANNERY AND ITS ENVIRONMENTAL EFFECT: BANGLADESH PERSPECTIVE

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ABSTRACT

The article has focused on the generation of ammonia from tannery especially in deliming operation by using conventional deliming agents such as ammonium chloride and ammonium sulphate. At Hazaribagh, Dhaka in Bangladesh every day approximately 220MT raw hides and skins are taking for the production of leather; after unhairing and liming (generally known as liming) pelt contents lime 1.2-4.4MT from where yearly 3.5×10^6–12.8×10^6 m^3 ammonia is produced; about 2.2×10^6 – 8.1×10^6 m^3 of ammonia is merged directly to atmosphere and about 1.3×10^6 – 4.7×10^6 m^3 of ammonia is discharged directly into wastewater at pH 8.5-9.0. The ammonia containing wastewater finally mixes to the river, Buriganga which causes aquatic problem. It has a negative impact on environment including human health effect, exposure of high concentration causes burning of nose, throat and respiratory tract. In tannery, workers are directly inhaling high concentration of ammonia during deliming as well as are suffering at a variety of difficulties. In atmosphere, ammonia reacts with available acids such as sulphuric acid (H_2SO_4), nitric acid (HNO_3) and hydrochloric acid (HCl) to form their corresponding salts causing cloudiness.

Keywords: Deliming, Ammonia, Ammonium Chloride, Ammonium Sulphate, Atmospheric.

1. INTRODUCTION

Ammonia is the main basic gas in a variety of important sample matrixes such as, ambient atmosphere, human breath and indoor air. Human skin also releases ammonia at a rate of few ng/cm^2 in 5 min (Nose *et al.* 2005). The hazardous nature of ammonia is reported by the US Department of Health and Human Services (USDHHS, 2004). It is reported that the historical Danish churches can be damaged by ammonia (Skytte *et al*. 2012). Mostly ammonia is released to atmosphere from a variety of biological sources. Due to industrialization, air is extremely contaminated by human activities, *e.g*. industrial process, burning of fuel, smoke, fireworks, burning of coal from where different types of gaseous pollutants *e.g*. oxides of sulphur (SOx), oxides of nitrogen (NOx), chlorine (Cl_2), oxides of carbon (COx), hydrogen sulphide (H_2S), ammonia (NH_3) etc. are continuously emitting and directly merge to atmosphere. Mark A *et al*. 2009 is reported that ammonia will be the largest single contributor by 2020 to each acidification, eutrophication and secondary particulate matter in Europe.

Worldwide tanning industry (tannery) is known as one of the most polluting and obnoxious industry due to generating solid, liquid and gaseous pollutants. Tannery in Bangladesh is an old industry; it is also generating high pollutants. After production, solid wastes are deposited nearby roadside; liquid wastes are discharged directly to drain and finally fall to the river Buriganga. On the other hand, gaseous pollutants like hydrogen sulphide (H_2S), ammonia (NH_3) and volatile organic compounds (VOCs) are emitted from unhairing and liming (generally known as liming), deliming and finishing operations respectively; all these pollutants are directly merged to atmosphere (Das H.K, 2000).

The 2nd chemical treatment of raw or wet salted hide/skin is known as liming. After liming, a fraction of lime content remains into pelt and it has to remove by conventional chemical treatment. During conventional deliming operation, huge amount of ammonia (NH_3) is produced; depending on pH value i) a fraction of ammonia is directly merged to atmosphere without any filtering and ii) the remaining fraction soluble into waste liquor is directly discharged through drain. Hence, ammonia conatinig waste liquor finally is mixed to the river water of Buriganga.

In atmosphere, ammonia is combined with NOx and SOx which normally are emitted from industrial and vehicle combustion processes and fine particulates are formed (Pope *et al*. 2002). These fine particulates are a contributor to smog in cites and decreased visibility in pristine areas. Smog is also a human health issue leading to an increased rate of respiratory and heart diseases (Ellis *et al.* 2011).

In Bangladesh there are about 270 leather industries, 243 out of 270 industries are located in the western part of capital city Dhaka, at Hazaribagh, covering the area of 25 ha; others are scattered all over the country (Ahmed M. 2012). Tannery at Hazaribagh, per day 220MT hides and skins are processed and discharged 7.70 million liters wastewater as well as 88MT solid waste (Ahmed K. A. 1997). In leather processing not only solid and liquid waste but also produced gaseous pollutants and discharged as green (without treatment) into the environment. Due to generating high pollutants as well as causing environmental pollution; leather industry in Bangladesh has gained a negative impact in the good tempered society therefore facing a severe challenge. But it is one of the most exports earning sector to strengthen the national economy of the country. The Export Promotion Bureau (EPB) reported that in the fiscal year 2011-12 Bangladesh earned US$765 million from the leather sector (Technical Report. 2013).

In this study, conventional steps for leather processing were monitored especially for deliming operation to estimate the generation of ammonia. The study also investigated the effect of ammonia i) on human health ii) on aquatic life iii) in atmospheric environment and iv) on vegetation.

Study Area

The study area is located at Hazaribagh, Dhaka, Bangladesh where 90% tanneries are running their poduction all the year round. The located area is shown in the Figure 1.

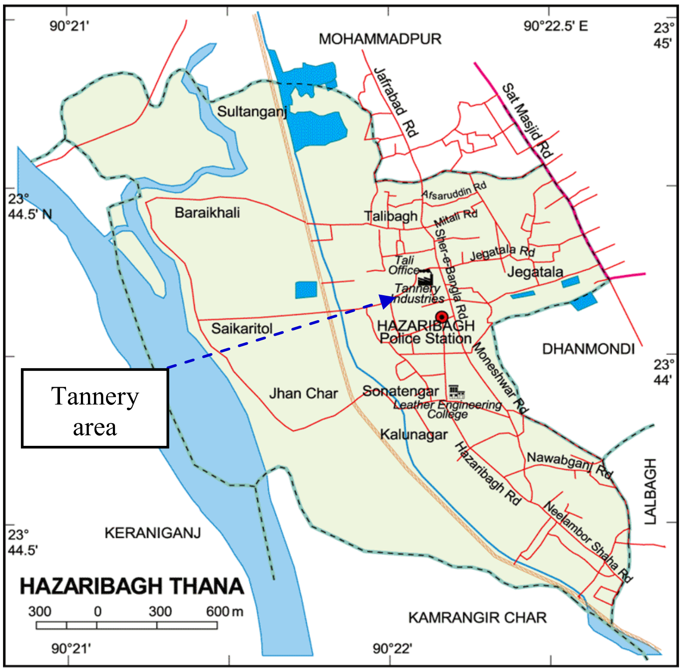

Figure 1: Tannery area l at Hazaribagh, Dhaka, Bangladesh.

2. METHODOLOGY

2.1 Sample and materials

Five big tanneries were selected to monitor conventional deliming process to estimate ammonia production. At Hazaribagh per day 220MT hides and skins are processed from where huge amount of ammonia is produced in deliming.

2.2 Chemicals and reagents

Worldwide numerous deliming agents are used such as acids (organic acids: formic acid, acetic acid, lactic acid; inorganic acids: hydrochloric acid, sulphuric acid, boric acid), acid salts (sodium metabisulphate, sodium metabisulphite) and ammonium salts (ammonium sulpahte, ammonium chloride, ammonium sulphamate) but only a few selected chemicals are used as deliming agents in Bangladesh. The deliming agents are used in Bangladesh as mentioned in the Table 1.

Table 1: Deliming agents are used in Bangladesh.

Reagents	**Manufacturers**	**Remarks**
Ammonium Chloride, NH_4Cl	Hainan Huarong, China	Frequently
Ammonium Sulphate, $(NH_4)_2SO_4$	BASF, Germany	Frequently
Sodium metabisulphite, $Na_2S_2O_5$	BASF, Germany	Moderately
Boric acid, H_3BO_3	BASF, Germany	Rarely
Hydrochloric acid, HCl	Awal & Brothers Ltd. Bangladesh	Rarely

2.3 Beamhouse operations

In tannery, it is the initial stage of leather processing. The stepwise processes namely pre-soaking, soaking, liming, deliming, bating and pickling are considered as beamhouse operation. The beamhouse operation has a great importance for leather production and deliming is the 4th chemical operation in beamhouse. The flow chart for beamhouse operations from wet salted hide or skin is shown in Figure 2.

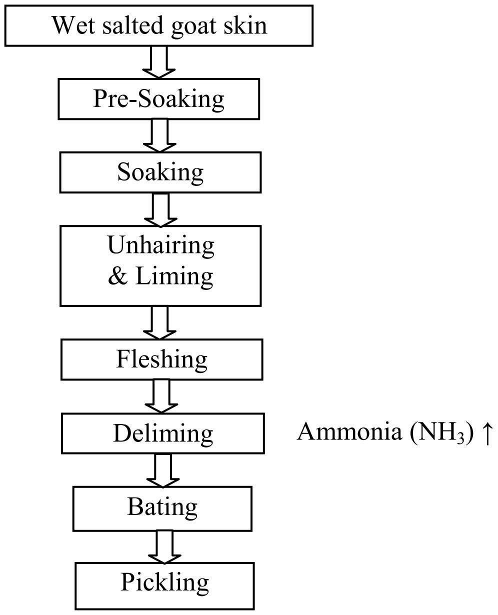

Figure 2: Flow chart for beamhouse operation

2.3.1 Deliming operation

Prior to deliming, hides and skins are treated with sodium sulphide and slaked lime for about 18-20hrs to remove hair and soluble protein is known as liming (actually unhairing and liming). In liming operation 4-5% lime is used from where pelt content lime is 0.5-2.0% by weight (Sarkar K. T., 2005).

Table 2: Deliming agents are used at selected tanneries

No.	Tannery*	% of Deliming agents	
		NH_4Cl	$(NH_4)_2SO_4$
1	A	0.25–0.5	2.0–3.0
2	B	0.30–0.5	1.5–3.0
3	C	0.25–0.4	2.0–3.0
4	D	0.25–0.5	1.5–2.5
5	E	0.35–0.4	1.5–2.5

*To maintain secrecy tannery name is undisclosed

This lime has to remove partially or fully based on the type of leather to be produced. In most cases, lime is removed fully by using the conventional deliming agents like ammonium chloride and ammonium sulphate. Sometimes boric acid is also used as experimental basis but not for commercial production because of competitive high price. Sodium metabisulphite is also used in combination with conventional deliming agents to obtain some bleaching effect on limed pelt. To remove lime from the pelt 1-2% ammonium sulpahte or ammonium chloride or combination of both are used as deliming agents based on pelt weight (Sarkar K. T., 2005). The conventional deliming agents are used at selected tanneries are shown in the Table 2.

2.3.2 Conventional deliming process

As stated, main objective of deliming is to remove lime from the pelt. There are numerous chemicals that can be used as deliming agnets to remove lime from the pelt. In conventional deliming process most commonly used deliming agents are ammonium chloride (NH_4Cl) and ammonium sulphate [$(NH_4)_2SO_4$]. After liming, pelt has to pass through an operation to reomve flesh and fatty substances known as fleshing. After weighing fleshed pelt, deliming process is done. The conventional deliming process in Bangladesh is shown in the Table 3**.**

Table 3**:** Deliming process (% based on *pelt weight)

Chemicals	%	Remarks
Water	60	Run 60 min
NH_4Cl	0.25–0.5	
$(NH_4)_2SO_4$	1.5–3.0	
After deliming float pH is 8.5-9.0		

*After liming hide/skin is treated as pelt.

$$\mathbf{Ca(OH)_2 + 2NH_4Cl = CaCl_2 + 2NH_3 + 2H_2O \text{ - - - - - } (1)}$$

$$\mathbf{Ca(OH)_2 + (NH_4)_2SO_4 = CaSO_4 + 2NH_3 + 2H_2O \text{ - - -- } (2)}$$

Based on (1) and (2) equations, ammonia is the common product in deliming process by the conventional deliming agents. Deliming to until chrome tanning drum rotates at 8 rpm. During drum rotating, ammonia is generated by chemical reaction. On the basis of pH **i**) a fraction of ammonia comes from the hollow axle of the drum and merges directly to atmosphere without any filtering and **ii**) remaining fraction is soluble into deliming waste liquor/wastewater which is released to drain to low lying land.

3. RESULTS AND DISCUSSION

3.1 Ammonia production in deliming

Equations (1) and (2) imply that one part of calcium oxide (CaO) can be neutralized by 2.4 parts of ammonium sulpahte or 1.9 part of ammonium chloride. In both cases equivalent amount of gaseous ammonia is produced. At Hazaribagh, per day 220MT hides and skins are processed and after liming pelt contents lime 1.2–4.4MT. To remove this lime from the pelt conventional deliming agents are used; it is estimated that 3.5×10^6–12.8×10^6 m^3 ammonia is produced yearly during deliming operation from tannery at Hazaribagh, Dhaka, Bangladesh. In the conventional deliming process pH is maintained 8.5–9.0 to facilitate the next operation known as bating. At this pH, the solubility of ammonia in water at 25°C is 15.3%–36.6% (Water Quality Guidelines for the Protection of Aquatic Life, 2010). Assuming 1.3×10^6 – 4.7×10^6 m^3 of ammonia is discharged directly into wastewater at pH 8.5-9.0 and remaining 2.2×10^6 – 8.1×10^6 m^3 is merged directly to atmosphere. The ammonia containing wastewater finally mixes to the river, Buriganga causing aquatic problem.

3.2 Effect on human health

Ammonia gas is a highly hydrophilic base that has irritant properties. While abruptly inhaling, it brings tears into the eyes. The effects of ammonia on human being are mainly due to its alkaline corrosiveness; either it's gaseous or liquid form can be irritating to the eyes, respiratory tract and skin (US DHHS, 2004). Ammonia and its hydroxide are corrosive, can rapidly penetrate to the eye and may cause permanent injury.

In tannery, operators are frequently handled the delimed pelt and waste liquor with bare hands and foot as well as without nose mask. Besides, persons who are engaged other works are exposed to inhale gaseous form of ammonia from inside the industry. As a result, person who are directly or indirectly involved in the tanneries are getting contact of ammonia or its hydroxide and suffering from many difficulties. It also causes headache, nausea and drowsiness.

3.3 Atmospheric effect of ammonia

Ammonia is lighter than air, as a result after emitting from any sources- it is merged directly to atmosphere. In deliming a part of ammonia ($2.2\times10^6 – 8.1\times10^6\ m^3$) un-dissolved at pH 8.5–9.0 is merged directly to atmosphere It has a short atmospheric lifetime of about 24 hrs (http://ammoniabmp.colostate.edu), it is a key atmospheric pollutant that could deteriorate ecosystem of health and contribute to respiratory problems in human. Once ammonia is emitted to atmosphere it could undergo conversion to NH_4^+ aerosol due to its highly reactive nature and quickly deposited near to the sources of emission (Ronald *et al.*, 1998). The conversion of ammonia (NH_3) to ammonium ion (NH_4^+) in aerosol or in clouds is dependence on the concentration of acids in atmosphere. It has been investigated by a group of researchers that ammonia catalyses the atmospheric oxidation of sulphur dioxide (SO_2) to sulphur trioxide (S_2O_3) and rapidly reacts with the acidic components in the atmosphere like sulphuric, nitric and hydrochloric acids to form the corresponding salts. The ammonium salts formed at atmosphere are the main components of smog aerosols; as a result, it affects on cloudiness of the atmosphere as well as the earth radiation budget (Jean *et al.*, 2004).

$$NH_3 + H_2SO_4 \longrightarrow NH_4HSO_4 + NH_3 \longrightarrow (NH_4)_2SO_4$$

$$NH_3 + HNO_3 \longrightarrow NH_4NO_3$$

$$NH_3 + HCl \longrightarrow NH_4Cl$$

Gaseous ammonia or particulate matters can be removed from the atmosphere by wet and dry deposition. Deposition of ammonia may occur: (i) as gaseous form (ii) in the aqueous form as ammonium ion (NH_4^+) and (iii) as an aerosol in submicron atmospheric water droplets. In the Figure 3 shows the flows of ammonia in the atmosphere. Once ammonia released into atmosphere, it returned to the earth surface either gaseous form or as an ammonium ion (Raven J. A. and Wollenweber B.1992). The ammonium ion associated with nitrate, sulphate or some other anion is incorporated into an aerosol or as part of the ionic mix is found in cloud and raindrops.

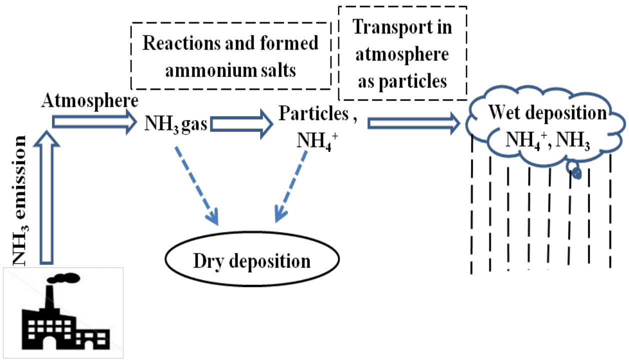

Figure 3: Emission of ammonia and its change at atmosphere

3.4 Effect on aquatic life

Ammonia is a colourless gas with a characteristic pungent smell. It is a toxicant to fish and other aquatic organisms. As it is dissolved in surface water, it exists in two forms: unionized (NH_3^0) and ionized (NH_4^+) and toxicity of unionized ammonia increases 10 times for increasing every unit of pH (SRAC Publication No. 463). Reported that ammonia is toxic to all vertebrates causing convulsions, coma and death, perhaps elevated ammonium ion (NH_4^+) previously ingested as unionized form (NH_3) displaces potassium ion (K^+) and depolarizes neurons. The depolarized neuron causes activation of *N*-methyl-D-aspartate (NMDA) type glutamate receptor; NMDA leads to an influx of excessive calcium ion (Ca^{2+}) and subsequently the central nervous system's cell is caused to death (Randall and Tsui 2002). In the fish pond, high concentration of ammonia is acutely toxic to fish may cause loss of equilibrium, hyper excitability, increased breathing, cardiac output, and oxygen uptake and in extreme cases cause to death (Stuart M. L., 2010). US EPA recommends an acute ammonia criterion of 2.9 or 5.0 mg N/L (for short-term exposure) and a chronic criterion of 0.26 or 1.8 mg N/L (for long-term exposure). In the tannery wastewater ammonia level is 50 mg N/L (Ganesh Chandra Saha and M. Ashraf Ali, 2001). In general, after completion of deliming operation, the waste liquor is discharged directly to drain to the low-lying area and finally falls to the river, Buriganga. As the aquatic livings like fishes intake unionized form of ammonia from wastewater; once got inside lately some converts to ionize form which causes cell damage. It also tends to block oxygen transfer from the gills to the blood as well as can cause gill damage (Joel Ogbonna and Amajuoyi 2010). Due to harmful effect of ammonium salts for the reproduction of fish, day by day numbers of fishes are decreasing in the wastewater contained ammonium salts in the low lying land adjacent of Hazaribagh as well as Buriganga River.

3.5 Effect on vegetation

Emission of ammonia from industrial or natural sources can be an additional source of nitrogen for plants; if the concentration is not so high. If the concentration is too high it may become toxic to plants. As mentioned, after dry or wet deposition ammonia affects on plant growth indirectly through change in soil pH. It reduces the pH of the solution or the soil. The effects of pH change depends on the type of plants as different plants prefer different pH ranges and some are better to acidic or neutral soils. It has been reported that nitrogen bearing salts could affect the soil ratios of nitrogen (N): phosphorus (P): potassium (K) (Hossain A. 2000). Some of the plants are becoming disappeared due to high concentration of ammonia vicinity to the tannery at Hazaribagh.

4. CONCLUSIONS

In deliming, yearly huge amount of ammonia is produced from tannery at Hazaribagh. The gaseous form of ammonia is merged directly to atmosphere where it reacts with available acids form their corresponding salts cause cloudiness and liquid form is finally discharged to river, Buriganga, caused various environmental problems. Whatever the form of ammonia either gaseous or aqueous has a negative impact on the environment including human health. All concerned authorities should take essential step to minimize the pollution from the tanneries or any other alternative way.

REFERENCES

Nose, K., Mizuno, T., Yamane, N., Kondo, T., Ohtani, H., Araki, S. and Tsuda, T. (2005). ‘‘Identification of Ammonia in Gas Emanated from Human Skin and Its Correlation with That in Blood.’’ 21(12), 1471-74.

“U.S. Department of Health and Human Services (2004), Public Health Service, Agency for Toxic Substances and Disease Registry.’’

Skytte, L., Rasmusen, K. L., Ryhl-Svendsen, M., Svensmark, Bo. and Brimblecombe, P. (2012). “Ammonia chemistry within Danish churches.’’ Sci. Total Environ. Vol. 417-418, 13-20

Mark, A. Sutton, Stefan R. and Samantha, M. H. Baker. (2009). “Atmospheric Ammonia, Detecting emission changes and environmental impacts.’’ ISBN 978-1-4020-9120-9, pp. 1.

Das H. K. (2000). “Chemical Toxicity and Environmental Hahzards of Tanneries in Bangladesh.’’ Framco-Bangladesh Association of Scholars and Trainees (FBAST). pp. 15-19.

Pope, C. A., Burnett, R. T., Thun, M. J., Calle, E. E., Krewski, D., Ito, K. and Thurston G. D. (2002). “Lung cancer, cardiopulmonary mortality, and long-term exposure to fine particulate air pollution.’’ J. Am. Med. Assoc. Vol. 287, 1132–1141.

Ellis, R. A., Murphy, J. G., Markovic, M. Z., Vanden Boer, T. C., Makar, P. A., Brook, J. and Mihele, C. (2011). “The influence of gas-particle partitioning and surface-atmosphere exchange on ammonia during BAQS-Met.’’ Atmos. Chem. Phys. Vol. 11, 133–145.

Ahmed M. (2012). “Effect of leather tanning industries on the environmental pollution and food chain.’’ Bangladesh AOTS Alumni Society (BASS). pp 17-19.

Ahmed K. A. (1997). “Surface water pollution from leather industries.’’ Bangladesh Leather. Vol. 11, 9-15.

Technical Report (2013): “Leather Sector Includes a Value Chain Analysis and Proposed Action Plans.’’

Sarkar K. T. (2005). Theory and Practice of Leather Manufacture, pp. 111.

Ammonia, Canadian Water Quality Guidelines for the Protection of Aquatic Life (2010). “Canadian Council of Ministers of the Environment.’’ http://ammoniabmp.colostate.edu

Ronald, B. McCulloch, Stephen, Fewa, G., George, C. Murray Jr. and Anejab Viney P. (1998). “Analysis of ammonia, ammonium aerosols and acid gases in the atmosphere at a commercial Hog farm in eastern North Carolina, USA.’’ Environ. Pollut. Vol. 102, 263-268.

Jean, J. R., Sherly, E. C., and Michael V. H. (2004). “Fate of ammonia in the atmosphere-a review for applicability to hazardous releases.’’ J. of Hazard. Mat. Vol. B108, 29-60.

Raven, J. A. and Wollenweber, B. (1992). “Ammonia and ammonium fluxes between photolithographs and the environment in relation to the global nitrogen cycle.’’ New Phytologist. Vol. 121, 5-15.

Durborow, R. M, Crosby, D. M. and Brunson, M.W. (1992). “Ammonia In fish ponds.’’ Southern Regional Aquaculture Center, SRAC 463.

Randall, D. J. and Tsui, T. K.N. (2002). “Ammonia Toxicity in Fish’’. Mar. Pollut. Bull. Vol. 45, 17-23.

Stuart M. Levit (2010). “A Literature Review of Effects of Ammonia on Fish.’’

Saha, G. C. and Ali, M. A. (2001). “Groundwater contamination in Dhaka from tannery waste.’’ 29(2), 151-66.

Joel Ogbonna F. and Amajuoyi Chinomso A. (2010). “Determination of the concentration of ammonia that could have lethal effect on fish pond.’’ ARPN J. of Eng. and App. Sci. Vol. 5, 1-5.

Hossain A. (2000). “Effluent Treatment of Leather Industries in Bangladesh.’’ Framco-Bangladesh Association of Scholars and Trainees (FBAST). pp. 30-35.

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GRAPHENE BASED REVERSE OSMOSIS DESALINATION TECHNIQUE: COST AND ENERGY PERSPECTIVE

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ABSTRACT

Growing concern of our limited water resources has led to renewed interest into Membrane Distillation of Reverse Osmosis (RO) as an alternative method of water purification and desalination. In contrast to other desalination techniques such as MSF and reverse MED, RO with grapheme membrane offers a potentially low energy and high rejection route to the desalination of highly dirty or salty waters. Feed water pressure consumption is also very low [28 bar] comparing with existing plants [56.1 bar]. Though the initial plant cost is high [about $40 million], it is 15-20% cheaper to run the traditional plants. Fresh water cost using grapheme based RO desalination plant is generally $4 to $6 per 1,000 gallons of water. Electricity cost is also cheaper As costing is a very crucial aspect for the developing countries like Bangladesh, this technology is the best way to salt rejection from water for its cost effectivity and high energy efficiency. Graphene has attracted conside rable interest over recent years due to its intrinsic mechanical, thermal and electrical properties. Incorporation of small quantity of graphene fillers into polymer can create novel nanocomposites with improved structural and functional properties. This review introduce about the new grapheme based RO system in water desalination, analytical assessment of this technique with the existing one, its low price and low energy use which is very much helpful to the developing countries people.

Keywords: Desalination, RO system, Graphene, Cost efficiency, Energy efficiency

1. INTRODUCTION

Clean water resources are rapidly being reduced around the world through human consumption, yet water is one of the most abundant elements on earth. The oceans represent the earth's major water reservoir. About 97% of the earth's water is sea-water while another 2% is locked in icecaps and glaciers. Available fresh water accounts for less than 0.5% of the earth's total water supply.Vast reserves of fresh water underlie the earth's surface, but much of it is too deep to access in an economically efficient manner. Additionally, seawater is unsuitable for human consumption and for industrial and agricultural uses. By removing salt from the virtually unlimited supply of sea-water, desalination has emerged as an important source of fresh water. Desalination is one of the most promising approaches to supply new fresh water in the context of a rapidly growing global water gap. But although oceans and seas contain about 97% of the world's water, desalination today only accounts for a fraction of a percent of the world's potable water supply. This is because existing commercial techniques for desalination suffer from important drawbacks, most importantly large energy footprints and high capital costs. A number of seawater desalination technologies have been developed during the last several decades to augment the supply of water in arid regions of the world. Salinity is also one of the major problems that the coastal region of Bangladesh has been facing over the last couple of decades. Due to sea level rise, frequent natural disasters, changes of climate patterns and man-made alteration of natural settings, the situation is becoming more vulnerable day by day. Seawater Reverse Osmosis (RO) is a very widely practiced desalination technique. Membrane technologies and reverse osmosis are increasingly used to augment municipal water supply, to produce high quality industrial water supplies, and to reclaim contaminated supplies.

One of the most sensitive and critical aspects of any water project is cost. Due to the constraints of high desalination costs, many developing countries like Bangladesh are unable to afford these technologies as a fresh water resource. Carbon has been the most versatile material used for water purification in history . The most widely used material for water purification today is activated carbon (AC) derived from plant sources. It has the best possible surface area and could be produced at low cost, making it the most affordable adsorption medium in diverse applications. A number of other forms of carbon have appeared with very large adsorption capacities. Today, Reverse Osmosis (RO) membranes are the leading technology for new desalination installation, for decreasing costs and producing superior water quality. But Reverse osmosis that now uses membranes to filter

the salt from the water, requires extremely high pressure and hence, energy use to force water through the thick membranes, which are about a thousand times thicker than graphene. Graphene is one of the fascinating additions into the carbon family, the one-atom thick sheets of carbon. It consists of a 2D sheet of sp2-bonded carbon atoms in a hexagonal honeycomb lattice which is the ultimate thin membrane. Potential advantages of graphene over existing RO members include negligible thickness (one or, several atomic layers) and high mechanical strength which may enable faster water transport, low pressure requirements, and a wide range of operating conditions than previously possible. This invention can be a better treatment for Bangladesh in the field of desalination. There are also many other new technologies that are upcoming, researchers are working hard to lower the production cost more and more.
In this paper, we report computational results indicating that use of single-layer graphene as desalination membrane in RO plants can be economically efficient & its efficiency is higher than any other diffusive RO membranes.

2. METHODOLOGY

2.1 Reverse Osmosis (RO) System

Reverse osmosis is a membrane separation process in which pure water passes from the high-pressure seawater side of a semipermeable membrane to the low-pressure permeate side of the membrane. To overcome the natural osmotic process, the seawater side of the system has to be pressurized to create a sufficiently high net driving pressure across the membrane.
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2.2 RO System Components

The two most basic individual components in a seawater RO system are the high-pressure feed pump and the RO membranes. These components comprise the heart of any RO system and require careful selection and application for successful operation. In addition to these, other components related to the pretreatment of the inlet water and adjustment of the product water are also included. As shown in Figure 1, an RO system consists of four major components.

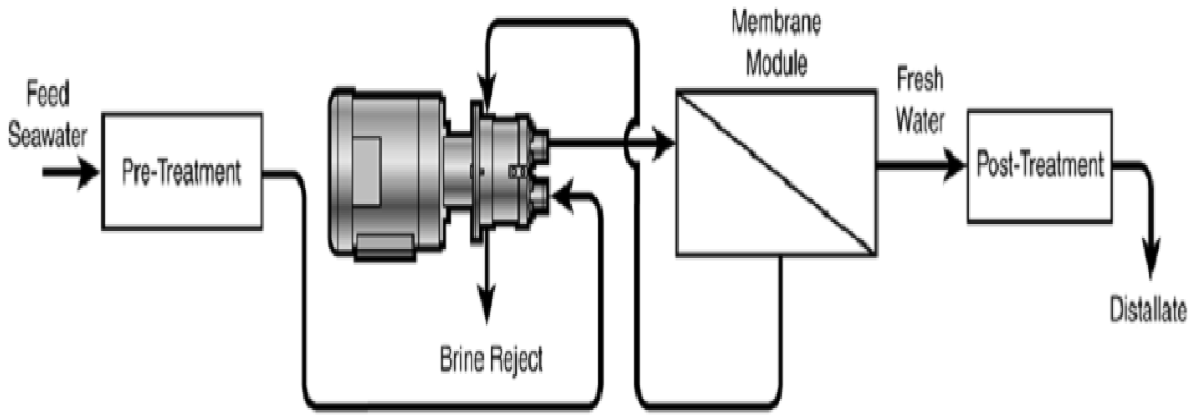

Figure 1. Schematic diagram of an RO system

2.3 Flow chart representation of RO system

RO treatment plant: Working with a feed pump and grapheme membrane

⇓

Pretreatment: removal of suspended solids, adjusting the pH

⇓

Pressurization: pump raises the pressure of the pretreated feed water

⇓

Separation: water is pressurized against the membrane using spiral wound module

⇓

Stabilization: pH adjustment and degasification are done before being transferred to the distribution system

2.3.1 Spiral-Wound module

A spiral-wound module element consists of two membrane sheets supported by a grooved or porous support sheet. The support sheet provides the pressure support for the membrane sheets, as well as providing the flow path for the product water. Each sheet is sealed along three of its edges, and the fourth edge is attached to a central product discharge tube. A plastic spacer sheet is located on each side of the membrane assembly sheets, and the spacer sheets provide the flow channels for the feed flow. The entire assembly is then spirally wrapped around the central discharge tube forming a compact RO module element.

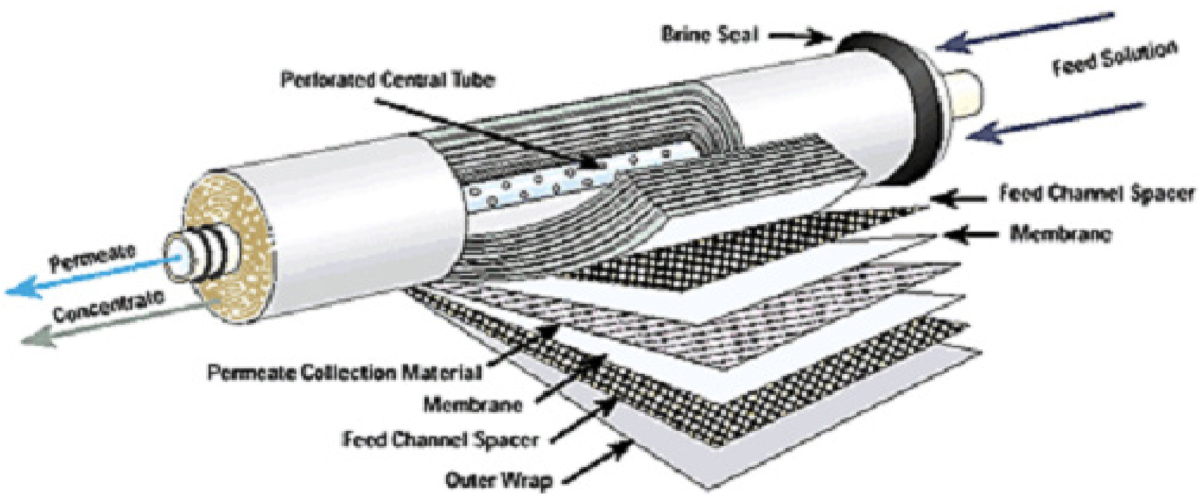

Figure 2. Spiral-wound RO module element

The currently used materials for seawater RO membranes are cellulose acetate membranes, polyamide membranes, and thin-film composite membranes. In this report we are introducing graphene as desalination membrane. Graphene is a flat monolayer of carbon atoms tightly packed into a two-dimensional

(2D) honeycomb lattice, and is a basic building block for graphitic materials of all other dimensionalities. It can be wrapped up into 0D fullerenes, rolled into 1D nanotube or stacked into 3D graphite (Figure 3).

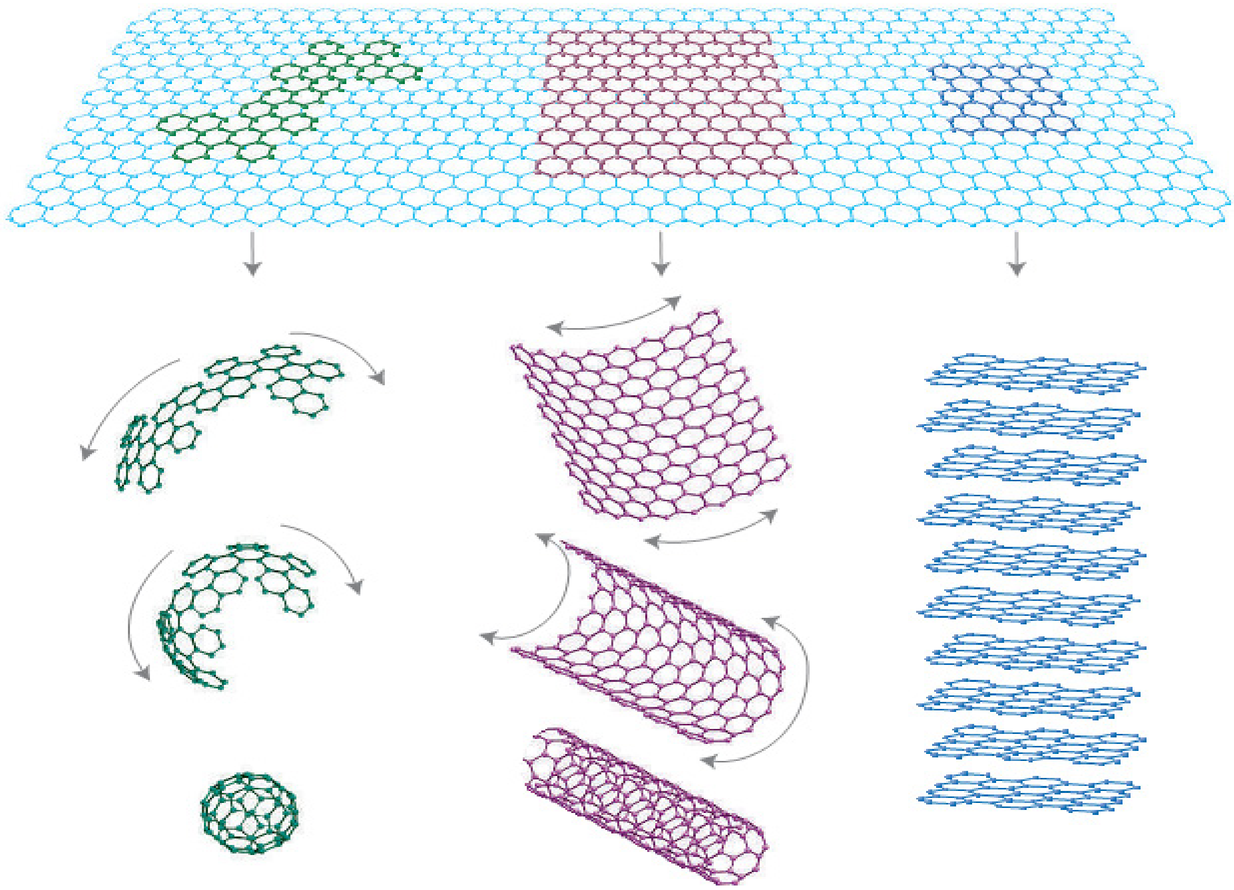

Figure 3. Mother of all graphitic forms

3. EFFICIENCY, COST ANALYSIS & RESULTS

3.1 Graphical representation

At the time of working with new technologies or thinking about new inventions, people always give preferences to 2 aspects, one is efficiency of new technology or something like that and another is its cost effectivity. We also give priority to these sides because these 2 is very much important for developing countries. At first,lets have a look on the costing of RO desalination plants according to its capacity:

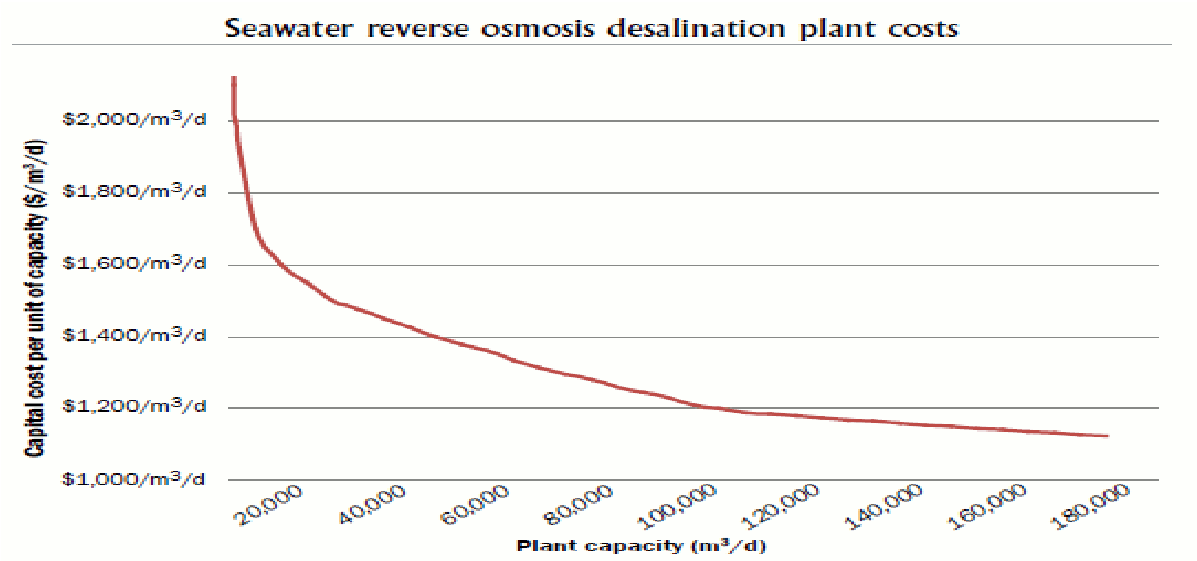

Figure 4: Seawater reverse osmosis desalination plant cost

From this graphical representation,it is clear to us that, desalination plant cost is very high with respect to the countries like Bangladesh. But it is worthy to say remind that, an MSF or MED plant with a capacity of 27,000 m3/day costs approximately $40 million.But an RO plant using membranes like graphene with a much higher capacity, approximately 100,000 m3/day, would cost around $50 million. So, initially a higher cost would be required for an RO plant, but the end result would be a significantly higher production rate. Now, we should discuss mainly about 'Graphene' as a distillation membrane, and its effectivity to use comparing with exsisting membranes. Graphene can be considered as an ideal membrane since its thickness is only one carbon diameter. Water can flow across a graphene membrane at rates in range of 10−100 L/cm2/day/MPa while still rejecting salt ions, which is 2 to 3orders of magnitude higher than diffusive RO membranes. Its maximum diameter is around 5.5 Å, that is, that the majority of salt ions approaching the pore entrance are able to pass through the membrane beyond this diameter. The flow rate of water is constant in time and increases with pore size and applied pressure. For narrow enough pores, water molecules are unable to pass and no water permeation is observed during the entire trajectory. Again, they would all be of the same size, so there would be no wasted space in the system, nor gaps so large that they let sodium and chloride ions through. The pores punched in this way are straight, rather than being convoluted channels as is the case in a polymer membrane. That also speeds the water molecules' passage. In contrast with classical RO membranes, where water transports slowly via a solutiondiffusion process, nanoporous membranes can allow for fast convective water flow across well-defined channels. RO is the most energy-efficient desalination technique to date with a record of 1.8 kWh/m3 recently achieved in a commercial plant (compared with an average ~ 5 kWh/m3 inthe 1990s). In terms of energy use, a graphene-based system could generate desalinated water at a far greater rate using the same amount of energy, or it could simply be run at lower pressure. Graphene sheets is also strong absorber, because of its balck color, it absorbs at the whole sun spectrum. Its advantages is also in chemical stability and low cost.

3.2 Tabular representation

Here is a overall comparision of grahene using plant with existing plants. The obtained water via desalination using graphene nanocomposite has been tested chemically and biologically. All chemical testing has been carried out according to the standard international protocols in order to determine water quality such as: Salinity, pH, dissolved oxygen test, hardness, heavy metal content, and the total amount of phosphate, sulphate and carbonate ions. All the result indicates that the obtained water is totally pure and free from all salts, metal ions and heavy metals. The filters are effective at removing bacteria also. Many protozoa, bacteria, viruses, algae and fungi are found in natural water systems. Some are pathagenic (typhoid, cholera and amoebic dysentry can result from water-borne pathogens). The excessive growth of algae (called 'algal bloom') can degrade water quality because it lowers dissolved oxygen levels thereby killing other living things. The level of bacterial contamination of water due to animal waste is measured by determining the number of coliform organisms such as E. coli. The obtained distillated water using this methodology was examined under optical microscopy and using Cell count reader in order to determine if there is any type of microorganism or bacteria exsit in our water sample. This biological testing indicate that the water is totally clean and it does not have any micro-organism.

That might be because of the photothermal effect of the nanocomposites which raise the temperature and cause bacteria death. Or because of the presence of silver nanoparticles which has super antiseptic and anti-bacterial activity.

Table 1: Comparison between RO Plant with Graphene & Others

BASIC PROPERTIES	**EXISTING RO PLANTS**	**RO PLANT WITH GRAPHENE**
INITIAL PLANT COST	$50 million [Capacity 100,000 m3/day	15-20% cheaper to run than traditional ones
POWER CONSUMPTION	297 Kwh/m3	295 Kwh/m3
FEED WATER PRESSURE	56.1 bar	28 bar
RECOVERY RATIO	0.42%	0.42%
ELECTRICITY COST	0.53- 0.06 US$/kWh	15-20% less than existing plants
FRESH WATER COST	0.82- 0.986 **$/m3**	Generally $4 to $6 per 1,000 gallons
PHYSIO-CHEMICAL PRINCIPLE	Solution-diffusion	Mainly solution-diffusion but can adopt other process
CHEMICAL CONSUMPTION	High	High
MEMBRANE REPLACEMENT	It is necessary in high rate	neccessary
TOLERANCE TO CHANGE IN SEAWATER COMPOSITION	Very low-low	Medium
MAINTAINANCE REQUIREMENT	High	High

Table 2: Obtained Water Quality via Desalination by Graphene

WATER QUALITY PARAMETERS	**OBTAINED DESALINATE WATER VIA GRAPHENE**	**NORMAL DRINKING WATER**
PH	7	6.8
Calcium content	0	50 ppm
Chlorine content	0	100 ppm
Oxygen	1.5%	2.2%
Sulphate content	0	20 ppm
Mg ions	0	30 ppm
Salinity	0	0.03 %
Hardness	0	2%

The development of nanomaterials like graphene not only provides many benefits to diverse scientific fields but also poses potential risks to humans and the environment. For the successful application of nanomaterials in water desalination, it is essential to understand the biological fate and potential toxicity of these nanoparticles. Biocompatible nanocomposites with grapheme was synthesized by chemical method. These data clearly indicates that the nanocomposites did not showed any cells toxicity for 6 hours incubation period even at relatively high concentrations (0.05 mM). However, at 18 hours incubation period, low toxicity (cells death) was detected at different concentration range from 5% to 15%. The results suggest that these Graphene nanocomposites do not have any cytotoxcity in the dark. RO system was introduced in 1970. In last few decades,it was developed in all aspects. Nowadays, using of various polymer membranes gives a new dimension to this desalination process There are now about 3,500 plants worldwide with a production capacity of about 3,000 mgd. As the demand for freshwater increases and the quality of existing supplies deteriorates, the use of desalination technologies will increase. Graphene as a distillation membrane will moderate the system more efficiently and its low price will be very helpful to the developing countries. On the other hand, energy efficiency is increasing day by day with the modernization of the technique and the using membranes. Here, a chart is represented showing the evolution of RO system, its modernization and less energy consumption:

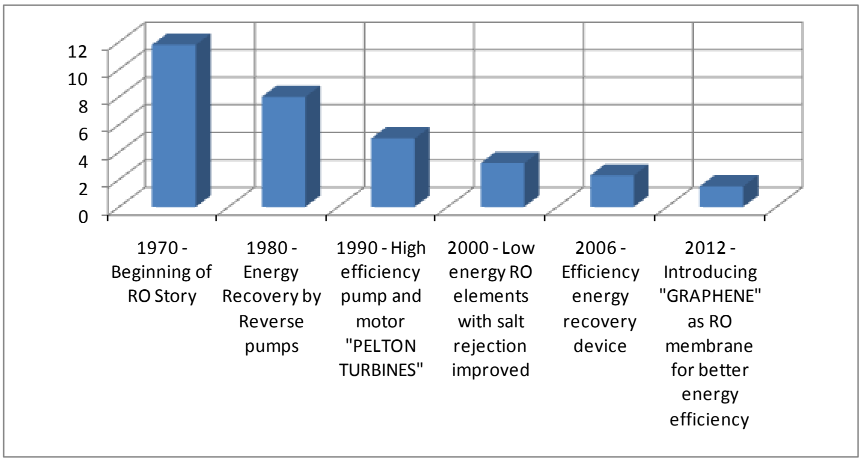

Figure 5: Gradual development of Graphene based membrane technique

4. CONCLUSIONS

Our report indicates that nonporous graphene membranes are able to reject salt ions while letting water flow at permeabilities several orders of magnitude higher than existing RO membranes. Due to the constrains of high desalination cost, many developing countries like Bangladesh who can not afford these technologies as a fresh water resource can adopt our proposed technique to meet their fresh water demand in an economical way. This work highlights the promise of atomically thin, periodic nanostructures like graphene for water desalination. Our approach strongly suggests that a bottom-up, systematic redesign of desalination membrane materials can yield significant improvements over existing technological methods. We expect that this work will add to the understanding of next-generation membranes for clean water technology.

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REFERENCES

David Cohen-Tanugi and Jeffrey C. Grossman. " Water Desalination across Nanoporous Graphene", Department of Materials Science and Engineering, Massachusetts Institute of Technology, Cambridge, Massachusetts 02139, United States.

Reza Dashtpour and Sarim N. Al-Zubaidy, "Energy Efficient Reverse Osmosis Desalination Process", International Journal of Environmental Science and Development, Vol. 3, No. 4, August 2012

Asmerom M. Gilau and Mitchell J. Small," Designing Cost-Effective Sea Water Reverse Osmosis System under Optimal Energy Options for Developing Countries", Proceedings of the International Conference on Renewable Energy for Developing Countries-2006

R. R. Nair, H. A. Wu1, P. N. Jayaram, I. V. Grigorieva1, A. K. Geim, "Unimpeded permeation of water through helium‐leak‐tight graphene‐based membranes".

Ramesh Valavala, Jinsik Sohn, Jihee Han, Namguk Her, Yeomin Yoon, " Pretreatment in Reverse Osmosis Seawater Desalination: A Short Review", Review Paper, Environmental Engineering Research.

C.Y. Tang, Y. Zhao, R. Wang, C. Hélix-Nielsen, A.G. Fane," Desalination by biomimetic aquaporin membranes: Review of status and prospects", Review of status and prospects, Desalination (2012).

A. K. GEIM AND K. S. NOVOSELOV, " The rise of grapheme", Manchester Centre for Mesoscience and Nanotechnology, University of Manchester, Oxford Road, Manchester M13 9PL, UK.

Soujit Sen Gupta, Theruvakkattil Sreenivasan Sreeprasad, Shihabudheen Mundampra Maliyekkal, Sarit Kumar Das, and Thalappil Pradeep," Graphene from Sugar and its Application in Water Purification", Research Article, ACS-Applied Material & Interfaces.

Ali Al-Karaghouli and Larry Kazmerski," Economic and Technical Analysis of a Reverse-Osmosis Water Desalination Plant using DEEP-3.2 Software", National Renewable Energy Laboratory Golden, Colorado 80401.

Alyson Sagle and Benny Freeman," Fundamentals of Membranes for Water Treatment"

Glueckstern, P.; Priel, M. "Potential cost reduction of seawater desalination", Water Science and Technology: Water Supply, 3 (2003) 39-47.

Ettouney, H. M.; El-Sessouky, H. T.; Faibish, R. S.; Gowin, P. J. "Evaluating the Economics of Desalination." In Chemical Engineering Progress, 2002; Vol. 98, pp 32-396.

Carlos Campos," The Economics of Desalination for Various Uses". CETaqua-Water Technology Centre.

CHARACTERISTICS AND MANAGEMENT OF COMMERCIAL SOLID WASTE IN KHULNA CITY OF BANGLADESH

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ABSTRACT

A massive volume of solid waste is generated every day in the city areas and unfortunately solid waste management is being deteriorated day by day due to the limited resources in handling the increasing rate of generated waste. Sustainable management for commercial solid waste is a concerning fact in Khulna city to lessen environment pollution and health problems which are also contribute to the climate changes. This study helps to scrutiny the existing management process and introduces a new proposal of management process to abate environmental pollution. The study has been conducted in the third largest city in the country Khulna, located in the southern part of Bangladesh to determine the generation rates, physical composition and characterization of commercial solid waste (CSW) and to identify the current situation of commercial solid waste management. A structured questionnaire was processed and waste collected from different waste generating sources was segregated and weighed. Commercial solid waste generation rate was found 0.44kg/person/day and an average commercial unit generated 3.93 kg of waste per day. In the generation of CSW, vegetable/food waste was highest (44%) followed by 17% of packaging material and 13% of plastic/polythene/rubber and lowest (2%) found as can/metal/tin. By weight, 51% of the waste was compostable in nature. It is necessary to take initiatives by both public and private sectors for effective management of waste.

Keywords: Commercial solid waste, Compostable, Commercial category, Generation, Management

1. INTRODUCTION

Bangladesh is a densely populated country; country's population will be about 17 cores by 2020 (Bahauddin & Uddin, 2012). In countries like Bangladesh solid wastes make an incredible environmental hazard and social problem in city lives. Solid Waste Includes any garbage, solid waste, sludge from a waste treatment plant, water supply treatment plant, or air pollution control facility and other discarded material, including solid, liquid, semisolid, or contained gaseous material resulting from industrial, commercial, mining, and agricultural operations and from community activities but does not include solid or dissolved material in domestic sewage or solid or dissolved materials in irrigation return flows or industrial discharges. Commercial waste is defined as the solid waste generated by stores, offices, institutions, restaurants, warehouses, and nonmanufacturing activities at industrial facilities (Tchobanoglous, 2003). The management of solid waste represents a major economic and environmental issue throughout the world. The ever increasing global concern on environmental health demands that wastes be properly managed and disposed of in the most friendly and acceptable way. This is to minimize, and where possible, eliminate its potential harm to humans, plants, animals and natural resources. Waste management in developing countries is of grave concern from two points of view. Firstly, the process of urbanization and population concentration that is inextricably linked to waste management issues is progressing at a pace that is much faster than was ever experienced by today's industrialized countries (DESA, 2011). The issue of waste management in developing countries, therefore, has emerged as a critical and impending disaster. Secondly, these countries often have difficulty in streamlining the institutional systems, administrative bodies, management capabilities and human resources that are needed to take the lead in solving solid waste problems (UNEP, 2009). It is thus very difficult for them to respond effectively to the newly emerging challenge of solid waste management. This is a situation that is common to many developing countries due to the negative legacy of long periods of colonial subordination, education problems and various other factors (JICA, 2005). The issue of poor solid waste management (SWM) has become a challenge for governments of developing countries in Asia and Africa (Calo` and Parise, 2009; Halla and Majani, 2003; Mwangi, 2000; Ogu, 2000; Zia and Devadas, 2008). Hence, this has huge consequences in terms of collection, disposal and the elimination of waste (Thonart *et al,* 2005; Moghadam *et al.*, 2009). Many experts from various cities in developing countries have expressed serious concern about improper waste treatment and disposal in these countries (Berkun *et al.*, 2005; Pokhrel and Viraraghavan, 2005; Barton *et al,.* 2008; Chung and Lo, 2008; Imam *et al.*, 2008; Sharholy *et al.*, 2008). In most developing countries, solid waste management is undertaken by the local authorities. These services include waste collection (either from households or district collection points) to final disposal. However, the low financial base and human resource capacity of these local authorities mean that in most cases they are only able to provide a limited service (Barton *et al.,* 2008). Inadequate management of solid waste in most cities of developing countries leads to problems that impair human and animal health and ultimately result in economic, environmental and biological losses (Wilson *et al.*, 2006; Kapepula *et al.,* 2007; Sharholy *et al.*, 2008). Khulna City is located in the southwestern part of Bangladesh near the Sundarbans, the largest mangrove forest in the world. It is situated below the tropic of cancer, around intersection of latitude 22.49º North and longitude 89.34º east. The total area of Khulna city corporation (KCC) is about 47 sq. km comprising 31 wards (BBS, 2001). The solid waste generation of the urban area in Khulna city is increasing proportionately with the growth of population, which is posing serious threat to the management and disposal of solid waste. Ethically, proper waste management is most important task of the city corporation for keeping the city clean and healthy. But municipal solid waste disposal by the city corporation is being an unmanageable burden to the city managers. The major problem of waste management, however, is the people's attitudes and behavior. Beside this, physical planning issues, government system and policies, and administrative and managerial procedures are considered as some of the major waste management problem in Khulna city (Hasan, 1998). The main motive of this study is to make a move in the systematic study of commercial solid waste management (CSWM), leading to quantification of the amount of waste generated from the residential area, determination of its composition, and to quantify the different composition characteristics of CSW.

2. METHODOLOGY

Extensive research works are needed in this field to find ways to abase the problems arising from improper solid waste management. A number of studies have been conducted in Bangladesh and few studies have also been conducted in Khulna on the issue of waste management. Firstly, a study involving the assessment of documents and records relating to commercial waste in Khulna City including Daulotpur Bazar, New market Bazar, Boyra Bazar, Fulbarigate Bazar has been conducted. Reconnaissance survey was conducted to identify the socioeconomic status and solid waste generation scenario of the study area, especially the sources and sub-sources from where the solid waste is generated, to observe the physical condition of the study area and to get information regarding quantity and quality of commercial solid waste. A structured questionnaire was designed, pre-tested, and modified to collect commercial solid waste related data. Based on the reconnaissance survey carried out, commercial units have been categorized into 11 major types. These 11 major types of commercial units have been selected according to the frequency of specific type in Khulna. Name of the major commercial categories mentioned below: a) Doctor's chamber and Pharmacy; b) Furniture gallery; c) Grocer's shop; d) Sanitary equipment shop; e) Vegetable market; f) Fish/meat market; g) Flower's shop; h) Refreshing corner; i) Stationary shop; j) Hotels/Restaurants k) Electrical shop. Eleven commercial units from each category have been randomly selected and thus a total of 100 commercial units have been studied out. During the questionnaire survey, polythene bags (similar size and with particular coding of the respondent) were supplied to each commercial unit to place their commercial wastes. Collected wastes from each commercial unit within the poly bag were weighed and recorded. Then the wastes within each bag were segregated and each segregated item was weighed separately and recorded. The same job was conducted each day for each of the 100 commercial units. During segregation, collected wastes from each bag were spread on clean plastic sheets and the wastes sorted by hand, following the methodology of Salam *et al.* (2012), Sujauddin et al. (2008) and Enayetullah *et al.* (2005).

3. RESULTS AND DISCUSSION

3.1 Commercial Solid Waste (CSW) generation

Cointreau (1982) has stated that the entire concept of waste is subjected to the value judgment of the primary owner or potential consumer. Broadly, waste can exist in any of three forms namely, solid, liquid, and gaseous or in all of three intermingled. In the specific case of solid waste, this can exist in either solid or semi-solid (i.e. sludge) forms. Present study addresses only solid waste. The analysis of the 100 sample observations in the study area indicates that an average commercial unit generated 3.93 kg of waste per day (Table 1). It also reveals that the rate of waste generation varies in different category of commercial unit studied. For this reason the waste generation rate by an average commercial unit has been minimum (1.2 Kg/day) by the electrical shop and maximum (15.40 Kg/day) by the hotels/restaurants.

Table 1: Commercial Solid waste generated by commercial unit per day

Name of the commercial category	Number of CU* studied	CWGR** (kg/CU/day)
Doctor's chamber and pharmacy	6	1.5
Furniture gallery	5	3.62
Grocer's shop	12	4.35
Sanitary equipment shop	7	1.23
Vegetable market	12	5.46
Fish/meat market	12	2.20
Flower's shop	6	3.27
Refreshing corner	8	1.36
Stationary shop	15	3.68
Hotels/Restaurants	6	15.40
Electrical shop	7	1.20
Total	96	3.93(Avg.)

*CU= Commercial Unit; **CWGR= Commercial Waste Generation Rate

The commercial waste generation rate per person in the study area is 0.37 Kg/day. The rate of waste generation per person varies in different category of commercial unit studied (Table 2). A total number of 289 staffs are engaged in 100 commercial units.

Table 2: Commercial Solid waste generation per person per day

Name of the commercial category	Number of staff studied	CWGR** (kg/person/day)
Doctor's chamber and pharmacy	18	0.16
Furniture gallery	25	0.12
Grocer's shop	31	0.37
Sanitary equipment shop	16	0.23
Vegetable market	29	0.92
Fish/meat market	35	0.22
Flower's shop	13	0.48
Refreshing corner	17	0.31
Stationary shop	40	0.23
Hotels/Restaurants	36	0.62
Electrical shop	29	0.45
Total	289	0.37(Avg.)

**CWGR= Commercial Waste Generation Rate

3.2 Physical composition of CSW generated by different commercial category

Composition of solid waste depends upon a number of factors such as food habits, cultural traditions, socioeconomic status and climatic condition (Enayetullah *et al*., 2005, Salam *et al.,* 2012). The physical composition of solid waste varies with different purposes of commercial categories (Table 3). All of the nine different items of waste (viz. paper, packaging materials, cans, plastic, textiles, glass, vegetable, rocks and wood) segregated during the study varied considerably among the commercial categories. Paper/ printed materials have been found the highest (18%) in Grocer's shop followed by 15% for Doctor's chamber. Packaging material was found highest 40% in Stationary shop followed by 15% in Grocer's shop. Plastic /polythene has been found maximum (27%) in both Stationary shop and Electrical shop. Vegetable/food waste found maximum (80%) in flower's shop followed by 65% in Fish/meat market and 63% in both vegetable market and Hotels/Restaurants. Textile has been found maximum (41%) in doctor's chamber. Glass/ceramic was recorded highest (48%) in sanitary equipment. Grass/wood/leaves have been recorded highest (55%) in furniture gallery. Table 3 also reveals that 51% of the waste has been compostable in nature by weight.

Table 3: Physical composition of CSW generated by different commercial category

Name of Commercial category	**Waste category (%)**								
	Non-compostable						**Compostable**		
	Paper	Pack	Plastic	Can	Glass	Rock	Vegetable	Textile	Wood
Doctor's chamber	15	12	17	6	3	6	0	41	0
Furniture gallery	11	0	4	8	0	8	9	5	55
Grocer's shop	18	15	13	2	0	3	41	8	0
Sanitary equipment	9	10	4	0	48	17	3	0	9
Vegetable market	2	3	19	0	0	4	63	6	3
Fish/meat market	0	0	24	0	0	7	65	1	3
Flower's shop	3	3	6	0	0	0	80	6	2
Refreshing corner	11	12	11	8	2	2	54	0	0
Stationary shop	12	40	27	3	0	6	3	5	4
Hotels/Restaurants	5	9	7	6	5	3	63	2	0
Electrical shop	5	14	27	0	15	4	0	10	25
Generation of waste/day by all category	7	17	13	2	4	6	44	4	3

3.3 Compostable and non-compostable waste

The quantity of Compostable and non-compostable waste of CSW generated in the study area is shown in Table 4. From the study it has been found the average compostable waste of CSW generation per commercial unit has been maximum (9.896 kg/day) for hotels/restaurants and minimum (0.750 kg/day) for refreshing corner and average non-compostable waste of CSW generation per institution maximum (5.504 kg/day) for hotels/restaurants and minimum (0.358 Kg/day) for sanitary equipment. Table 4 also reveals that the compostable waste of CSW generation by a person in the study area maximum (0.83 Kg/day) for vegetable market and minimum (0.04 Kg/day) for furniture gallery and average non-compostable waste of CSW generation by a person maximum (0.34 kg/day) for electrical shop and minimum (0.07 Kg/day) for doctor's chamber.

Table 4: Quantity of compostable and non-compostable waste generated by commercial categories

Name of Commercial Category	Compostable		Non-compostable	
	kg/CU/day	kg/person/day	kg/CU/day	kg/person/day
Doctor's chamber	0.874	0.09	0.626	0.07
Furniture gallery	3.137	0.04	0.483	0.08
Grocer's shop	2.538	0.23	1.812	0.14
Sanitary equipment	0 .872	0.09	0.358	0.14
Vegetable market	3.338	0.83	2.122	0.09
Fish/meat market	1.648	0.14	0.552	0.08
Flower's shop	2.190	0.32	1.080	0.16
Refreshing corner	0.750	0.19	0.610	0.12
Stationary shop	2.410	0.14	1.270	0.09
Hotels/Restaurants	9.896	0.46	5.504	0.16
Electrical shop	0.30	0.11	0.900	0.34

3.4 Existing & Proposed Management Process

According to KCC, approximately 620 tons of solid waste is generated in Khulna city every day. However, only 370-380 tons of the generated waste is collected daily by KCC and reaches the municipal disposal site. Therefore, between 250 and 260 tons of waste remains uncollected every day. Some of this is recycled by the informal recycling industries that exist in the area. The remaining uncollected waste is often dumped in an uncontrolled manner throughout the city, clogging drains, blocking roads and occupying vacant plots of land. The existing management process in the study area is shown by flow diagram in Figure 1.

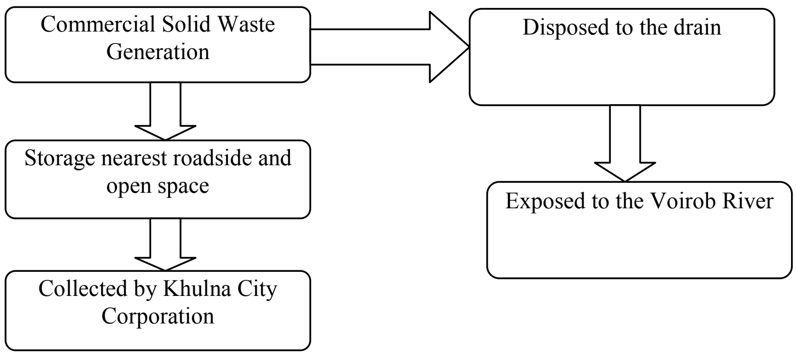

Figure 1: Existing management process of commercial solid waste

In order to deal with the prevailing situation in a planned way, intensive study is required to analyze the waste management scenario. New proposed management process progressed by the following diagram in Figure 2.

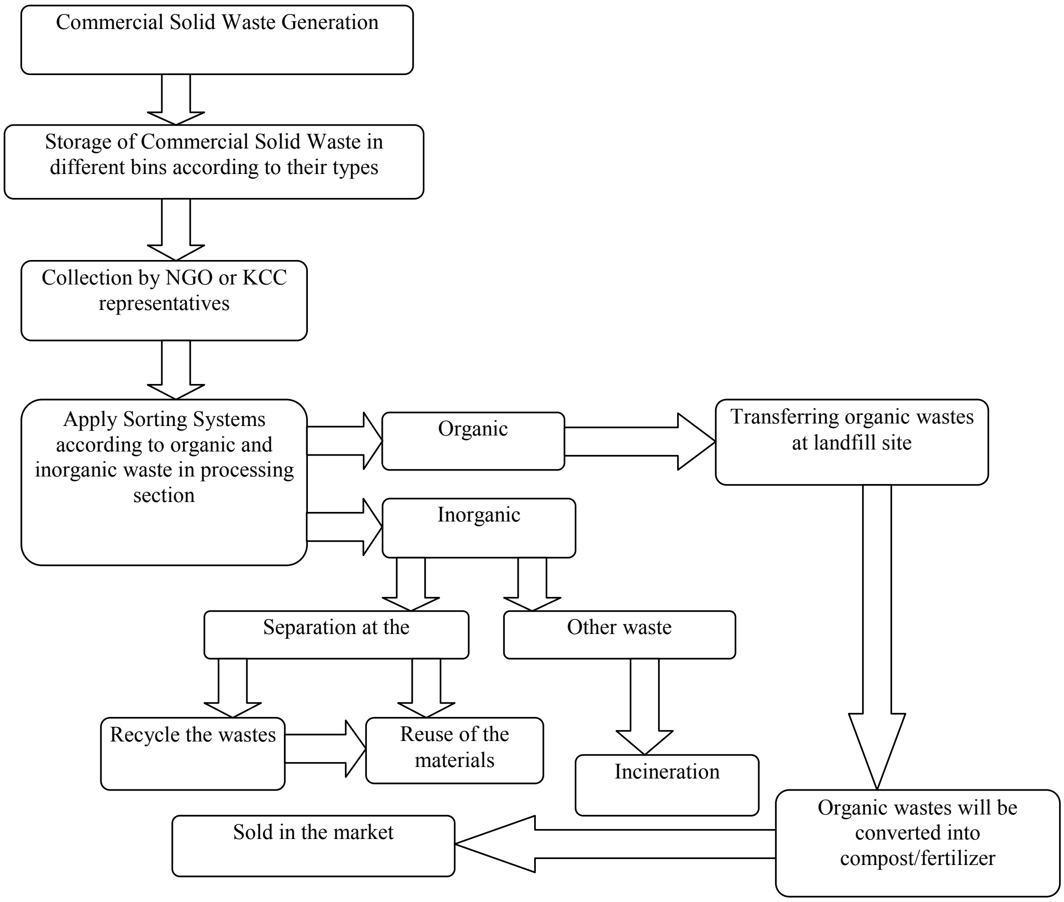

Figure 2: Proposed management process of commercial solid waste

In this process, firstly generated commercial waste are recognized and these are stored separately in different bins according to their types such as plastic bins, glass bins etc. Different colors bins are used for storage. It is helpful to differentiate the storage materials. Sorting systems according to organic and inorganic waste are applied in processing section after proper collection by KCC or local NGO. Organic wastes are properly transported to landfill site for converting into compost/fertilizer and after processing deliver to the consumer. Recyclable materials of inorganic wastes such as newspaper, bottles, plastic, containers, broken glass, cans, polythene etc. are prefer in recycling process and after recycling reuse the materials. Everyday about 1.8 tons of old plastic bags (bags of cement, rice, fertilizer) were handled in different SRM in Khulna city. Some used bags were processed separately to produce different sizes of shopping bags for further use. The quantity of books, papers and mixed papers that were reused every day were 0.85, 2.07 and 3.32 tons, respectively. Other materials of inorganic waste which are not recyclable are incinerated at incineration plant. Incineration option for waste management emits large amount of CO_2 which plays vital role in global warming. On the other hand, anaerobic digestion might be another option which emits considerable amount of CH_4.

4. CONCLUSIONS

Commercial solid waste has been increasing rapidly in Khulna City. The main contributing factors to the increasing waste generation rate are the urban population and GDP growth. There is a seasonal fluctuation in waste generation: higher waste generation rate occurs in the wet season and lower waste generation occurs in the dry season. The average CSW generation per capita is 0.37 kg/person/day, which is close to many developing countries. This paper has provided some qualitative and quantitative information on commercial solid waste, such as waste generation and its physical properties and characteristics. This paper also focuses on weak points in the criteria used by pertinent studies for the storage of commercial solid wastes. To ensure better human health and safety of workers involving in the process of waste disposal, effective solid management system is needed and it must be economically sustainable. This study tried only to unfold a theoretical model for better commercial solid waste management in Khulna city. To investigate the possibility of this model, a complete empirical study is necessary. This study will also prepare the platform for additional study and exploration of the commercial solid waste management.

REFERENCES

Bahauddin K. M. and Uddin M. H. (2012). “Prospect of Solid Waste Situation and an Approach of Environmental Management Measure (EMM) Model for Sustainable Solid Waste Management: Case Study of Dhaka City.” J. Environ. Sci. & Natural Resources, Vol. 5(1), 99 – 111.

Barton JR, Issias I, Stentiford EI. (2008). “Carbon-making the right choice for waste management in developing countries.” Waste Management, Vol. 28, 690–8.

Berkun M, Aras E, Nemlioglu S. (2005). “Disposal of solid waste in Istanbul and along the Black Sea coast of Turkey.” Waste Management, Vol. 25(8), 847–55.

BBS (2001). Bangladesh Bureau of Statistics Pocket Book. Dhaka, Bangladesh.

Calo F, Parise M. (2009). “Waste management and problems of groundwater pollution in karst environments in the context of a post-conflict scenario: the case of Mostar (Bosnia Herzegovina).” Habitat International, Vol. 33(1), 63–72.

Chung SS, Lo CWH. (2008). “Local waste management constraints and waste administrators in China.” Waste Management, Vol. 28(2), 272–81.

Cointreau, S. J. (1982). “Environmental Management of Urban Solid Wastes in Developing Countries – A Project Guide.” The World Bank, Washington, DC.

DESA (2011). “World Urbanization Prospects.” Department of Economic and Social Affairs/Population Division, United Nations, New York.

Enayetullah I, Sinha AHMM, Khan SSA. (2005). “Urban Solid Waste Management Scenario of Bangladesh: Problems and Prospects.” Waste Concern Technical Documentation, Dhaka, Bangladesh. 18.

Halla F, Majani B. (2003). “Innovative ways for solid waste management in Dar-Es-Salaam:toward stakeholder partnerships.” Habitat International, Vol. 23(3), 351–361.

Hasan S. (1998). “Problems of Municipal Waste Management in Bangladesh: An Inquiry into its Nature.” Habitat ITNL, Vol. 22(2), 191-202.

Imam A, Mohammed B, Wilson DC. Cheeseman CR. (2008). “Solid waste management in Abuja, Nigeria.” Waste Management, Vol. 28(2), 468–72.

JICA (2005). “Supporting Capacity Development in Solid Waste Management in Developing Countries – Towards Improving Solid Waste Management Capacity of Entire Societies.” Japan International Cooperation Agency.

Kapepula KM, Colson G, Sabri K, Thonart P. (2007). “A multiple criteria analysis for household solid waste management in the urban community of Dakar.” Waste Management, Vol. 27, 1690–705.

Moghadam MRA (2009), Mokhtarani N, Mokhtarani B. Municipal solid waste management in Rasht City, Iran. Waste Management, 29(1), 485–9.

Mwangi SW (2000). Partnerships in urban environmental management: an approach to solving environmental problems in Nakuru, Kenya. Environment and Urbanization, 12(2), 77–92.

Ogu VI (2000). “Private sector participation and municipal waste management in Benin City, Nigeria.” Environment and Urbanization, Vol. 12(2), 103–117.

Pokhrel D, Viraraghavan T. (2005). “Municipal solid waste management in Nepal: practices and challenges.” Waste Management, Vol. 25, 555–62.

Salam MA, Md. Lokman Hossain, Das SR, Wahab R, Hossain MK. (2012). “Generation and Assessing the Composition of Household Solid Waste in Commercial Capital City of Bangladesh.” International Journal of Environmental Science, Management and Engineering Research, Vol.1(4), 160-171.

Sharholy M, Ahmad K, Mahmoud G. Trivedi RC. (2008). “Municipal solid waste management in Indian cities.” Waste Management, Vol. 28(2), 459–67.

Sujauddin M, Huda SMS, Hoque ATMR. (2008). “Household solid waste characteristics and management in Chittagong, Bangladesh.” Waste Management, Vol. 28, 1688–1695.

Tchobanoglous G. (2003). “Solid Waste Management.” Environmental Engineering, fifth edition, Edited by Joseph A. Salvato, Nelson L. Nemerow, and Franklin J. Agardy, 755-758.

Thonart P, Diabaté SI, Hiligsmann S, Lardinois M. (2005). « Guide pratique surlagestion desdéchets ménagers etdes sites d’enfouissement technique dans les pays du sud. » IEPF, Canada.

UNEP (2009). “Developing Integrated Solid Waste Management Plan, Training Manual.” ISWM Plan, United Nations Environmental Programme, Osaka/ Shiga, Japan, Vol. 4.

Wilson DC, Velis C, Cheeseman C. (2006). “Role of informal sector recycling in waste management in developing countries.” Habitat International, Vol. 30, 797–808.

Zia H, Devadas V. (2008). “Urban solid waste management in Kanpur: opportunities and perspectives.” Habitat International, Vol. 32(1), 58–73.

FEASIBILITY OF ROOFTOP RAINWATER HARVESTING IN COASTAL AREAS: A CASE STUDY

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ABSTRACT

In the coastal belt region of Bangladesh suitable water sources are scares. The coastal belt suffers from high salinity in groundwater sources for the development of water supply system. In this situation, rainwater harvesting may be a better solution as an alternative solution. The coastal and hilly regions of Bangladesh usually receive heavy rainfall during the rainy seasons and little in dry periods. In this study, a pilot project on rooftop rainwater harvesting (RWH) was undertaken in the Khulna University of Engineering & Technology (KUET) campus to evaluate its feasibility for large scale application in the coastal areas of the country. The potential rainwater harvesting scheme, having an estimated 27000 m^2 of rooftop catchment area, was found adequate to fulfill the drinking water demands for the present population of 4550 persons in KUET campus. The shortage of rainwater in dry periods would be supplemented by storing surplus water during the rainy season. From available data, it was found that the average annual rainfall in Khulna region was approximately 1800 mm/yr. The pilot study has been carried out in Bldg-14 at KUET campus with an expected total population of 30 persons. The required drinking water storage volume was determined 13.62 m^3 and 17.73 m^3 using 'Mass Curve Method' and 'Area Consumed and Volume Consumed (Ac-Vc) Method', respectively.

Keywords: Coastal areas, feasibility study, rain water harvesting, rooftop catchment, sand filtration

1. INTRODUCTION

Bangladesh is a deltaic country with a total land area of 147,570 km^2 and a population over 156 million (CIA Factbook, 2010). It is often called the 'land of rivers' where more than 700 rivers and their tributaries formed a large network of hydro-system that has a length of 21,140 km. The three major rivers of Bangladesh, well known as The Padma, Meghna and Jamuna, which bring large flow of water and the golden alluvium mostly from the Himalayan-Tibetan plateau, play a lifeblood role in its economy and culture. However, continuous exploitation of river water system due to man made changes, industrial pollution and damping of household wastes, most of the river has lost its quality. In many points, the country's main river Padma has lost its natural water flow that dried out the tributaries and started the process of desertification.

In Khulna, only 20 percent of the urban area has been covered by public water supply system. Water supply in Khulna relies on groundwater. Groundwater is being depleted in many areas has a critical water supply system

Groundwater levels are at or near ground level during the period of August to October and lowest in April to May. The availability of potable ground water has become a problem for the following reasons:

- Presence of Arsenic and Dissolved ion
- Salinity in the coastal area
- Lowering of ground water level
- Rock, stony layers in hilly areas

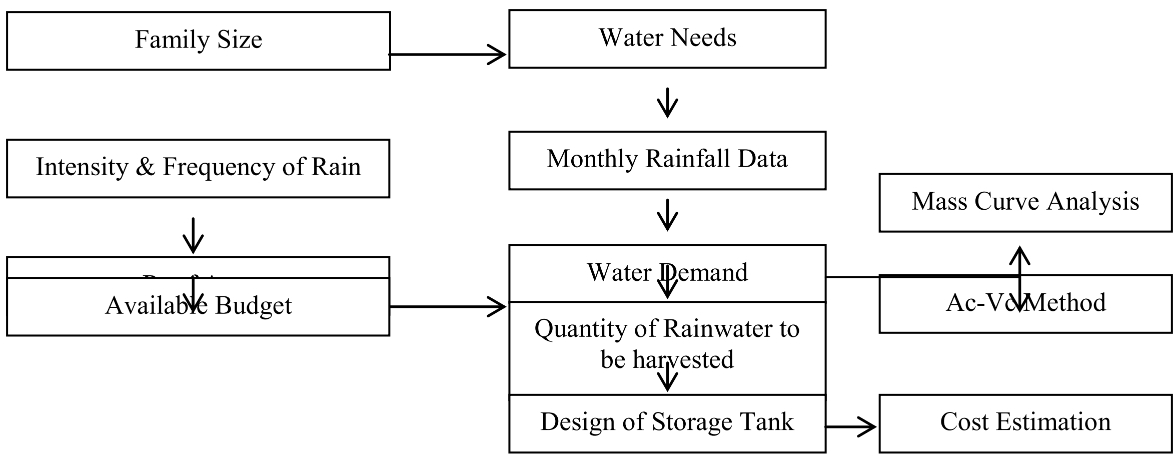
In the face of increasing scarcity of water resources, there is a need for communities especially in coastal region where there is scarcity of drinking water due to high salinity to undertake audits to their current rainwater harvesting technique as a practical and promising alternative solution of water storage. Rainwater harvesting involves water collection from surface like rooftop on which rain falls and storing it for later use.

In this situation, rainwater harvesting may be a better solution to an alternative solution. In this paper, a feasibility study of rainwater harvesting at KUET campus is investigated.

2. METHODOLOGY

2.1 Research Strategy

The methodology of this study starts with the water needs for the community on which the rainwater harvesting system is applied is needed to be estimated. Thereby, the family size and the alternative water sources are needed to know. From the previous latest rainfall data, the annual average rainfall and the average monthly rainfall data should be collected. This is required to know the duration of dry period on which the storage option would be selected and estimated. Water demand for the selected community has been calculated by two methods, a) Mass Curve Analysis, b) Ac-Vc Analysis. Then the quantity of water to be harvested can be estimated. This process was followed by the design of storage option which depends on the intensity and frequency of rain, size of the roof surface, available budget and available material and labor. Finally the total cost of whole rainwater harvesting system is determined. Flow chart of the research strategy is given below:



2.2 Rainfall Pattern in Khulna City

As Rainfall is not uniform in Khulna city as well as all over the Bangladesh over seasons, it is quite adequate for rainwater harvesting. Annual average rainfall of Khulna is 1800 millimeters (mm) approximately 90% of the annual average rainfall occurs between June and October. Table 1 shows the maximum rainwater storage period should be fixed for at least 4~6 months from monthly rainfall data.

Table 1: Mean Monthly Rainfall (mm) in Khulna City (World Weather Information Service, 2012)

JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
0	0	0	0	0	24	255	24	84	6	0	0

2.3 Storage Option and Requirement

It is quite known that precipitation intensity is not uniform all over the year. But, excess rain stored during monsoon period may solve the crisis in the dry period by proper storage. The rainfall received by catchment area (roof and ground) is partly discharge towards the storage tank. A portion of rainwater is serving to wet the surface of the catchment area, and then lost by evaporation or infiltration in the ground. Generally, water captured during first 10 minutes of rainfall during an event of average intensity is unfit for drinking purpose. The amount of water can be yielded per month by a catchment may be computed as,

$$\text{Yield} = (f \times A \times R)/1000 \ m^3/\text{month} \tag{1}$$

Where, f= Catchment efficiency factor of runoff coefficient

A= Catchment area in m^2

R= Monthly rainfall in mm

2.4 Water Demand Analysis Method

2.4.1 Mass Curve Analysis

Analysis of rainfall data is involved in mass curve analysis. Success of this analysis depends on at least 10 years raw data. This include tabulating the monthly rainfall (mm) data, Monthly supply (liters) , cumulative supply (liters), monthly demands (liters), monthly amount of water stored (liters) and total amount stored (liters).

The required storage volume can be determined when the table will filled out. The least amount of stored during the dry season is subtracted from the largest amount stored during the wet season.

The tank size necessary for storage of rainwater is the difference between the maximum surplus of water in the rainy seasons and the maximum deficit in the dry season as determined by the mass curve method for the year.

2.4.2 Area Consumed (Ac)-Volume (Vc) Relation Method

In this method, a relation between critical catchment area per person (m^2/capita) and minimum storage volume provided per person served (m^2/capita) is determined based on a series of N year actual monthly rainfall data and a selected frequency of periods with limited supply. This method requires a series of calculation using different formulas as follows:

$$Ac = Ac_{min} = \frac{C\times 12}{f\times Rmin}\ m^2/capita \quad (2)$$

Where,
Ac_{min} = Minimum catchment area, m^2/capita
C = Monthly demand per capita, liter/capita
R_{min} = Lowest annual rainfall over the observed period, mm/year
f = Runoff coefficient taken as 0.75 (concrete roof)

$$R_C = \frac{C}{f\times Ac}\ mm/month \quad (3)$$

Where,
Rc = Amount of rainfall required per month to fulfill the required consumption (i.e., critical rainfall), m^2/capita.

$$D_C = \frac{Ac\times f\times(Rc\times Nc-\sum Ri)}{1000},\ m^3/capita \quad (4)$$

Where,
Dc = Deficiency (supply-demand) in a year, m^3/capita
Nc = Number of dry period (month) in a year
Ri = Rainfall during the dry period, mm/month

$$Vc = \frac{Dc}{1-Lc\times 0.5\times Nc}\ m^3/capita \quad (5)$$

Vc = Storage capacity, m^3/capita (shown in appendix B)
Lc = Loss factor (1/month)

2.5 Cost Analysis

Cost analysis for different RWH techniques as well as different storage option is an important factor. The most important is the cost of storage tank as it requires 80~90% of costs of RWH system. As the storage tank can be plastic, metallic, concrete or brick structure depending upon the suitability with space, environment and financial condition. For the pilot study for building 14 in KUET campus, the concrete or brick structure requires in case of suitability. Thereby, cost analysis of these methods is essential.

3. RESULTS AND DISCUSSION

The monthly rainfall patents in Khulna city were examined to have an idea about the adequacy of rainwater harvesting. The drinking water demand of forecasted population in KUET campus and potential rainwater harvesting is also described in this chapter. Generalized demand vs. supply curve to determine required storage volume and reliability chart also determined. The filtration method of rainwater by slow sand filter is also focused in this chapter. Storage volume required to cover the drinking water demand in dry season is the key part of analysis the rainwater harvesting system for a sustainable water supply system.

3.1 Estimated Population in KUET Campus

There are six student halls in KUET campus and eighteen residential buildings. During office period some people come from outside the university to join their woks and leave the campus in evening. Total residential population of KUET campus was found approximately 3480 person and population those are only available in peak periods was found approximately 300 persons.

Table 2: Residential Population of KUET Campus

Department	Population
Total population in teachers building	550
Total population live in dormitory	50
CE	480
EEE	480
ME	480
CSE	480
IEM	240
ECE+LE+URP	720
	Total = 3480 people

Table 3: Population available only in day time in KUET campus

Department	Numbers of Employee
Administration building	60
CE building	50
EEE building	40
CSE building	40
ECE building	40
IEM building	40
Engineering section	30
	Total = 300 people

Total population = (3480+300) * 1.2 = 4536 = 4550 person (say)

3.2 Estimated Building Roof Area in KUET Campus

The total building roof area in KUET campus was estimated 27000 sq. meters and the roof were constructed as concrete paved surface that is suitable as a rainwater harvesting catchment. The whole roof area might not be effective as rainwater catchment and thus a reduction factor 0.7 was taken on the total roof area. The map of different buildings and their tabulation form for roof area determination in various buildings at KUET campus is given bellow:

Table 4: Roof Area of Different Building in Study Area (KUET campus)

Building Name	Area (m^2)
B1	394
B2,B3,B4	143
B5,B6	340
B7,B8,B9,B10	316
B11,B12,B13,B14	250
B15	226
B16,B17	234
B18	185
EEEB	1104
CEB	1262
MEB	953
SB	953

CLB,CSEB,ECEB,EIMB	870
SCHOOLS	650
G HOUSE	232
DORMITORY	510
WORKSHOP	1860
HIT ENGIN	446
ADMINISTRATION BUILDING	641
STUF CLUB	259
ROKEYA HALL	900
RASHID HALL	1347
FAZLUL HAQUE HALL	1349
KHANJAHAN ALI HALL	1486
LALON SHAH HALL	1208
EKHUSEY HALL	2790
BONGOBONDHU HALL	2790
TOTAL =	27000

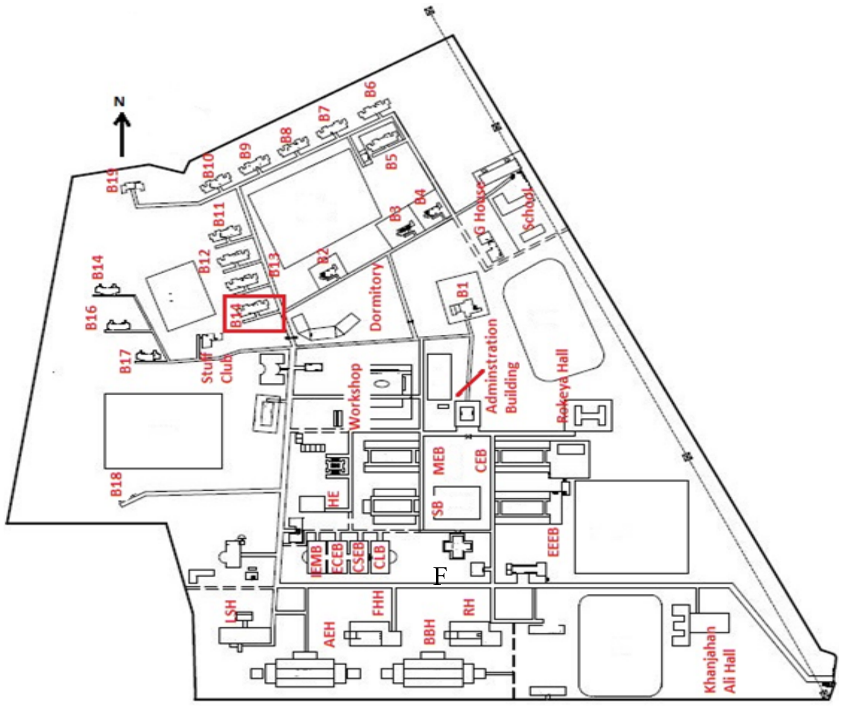


Figure 1: Existing Master Plan of KUET Campus

3.3 Rainfall Pattern in Khulna City

Rainwater is available in adequate quantity in Bangladesh during the monsoon periods. A 20 year rainfall pattern based on the mean intensity recorded in Khulna station for the period 1988 to 2007 is shown in Figure 2. It shows that the average yearly rainfall in Khulna city during 1988-2007 is 1800 mm which means that about 1.8 m^3 rainwater is available per m^2 of roof area each to develop a sustainable drinking water supply. In Khulna division, located in the coastal region of Bangladesh with an immense fresh water problem, rainwater harvesting would be an alternative promising option as a sustainable source for drinking water.

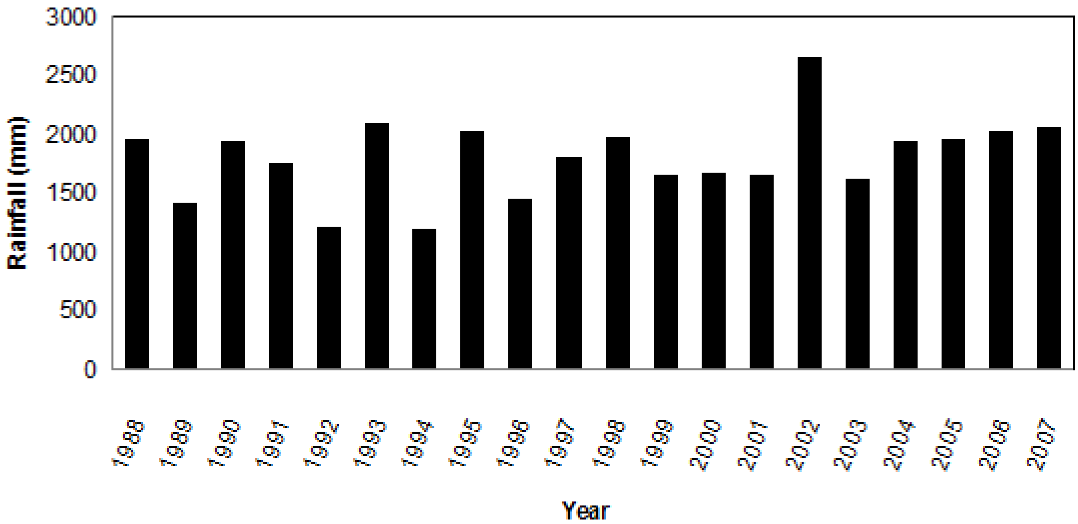


Figure 2: Variation of Annual Rainfall in Khulna city of 20 year data. (Bangladesh Metrological Department)

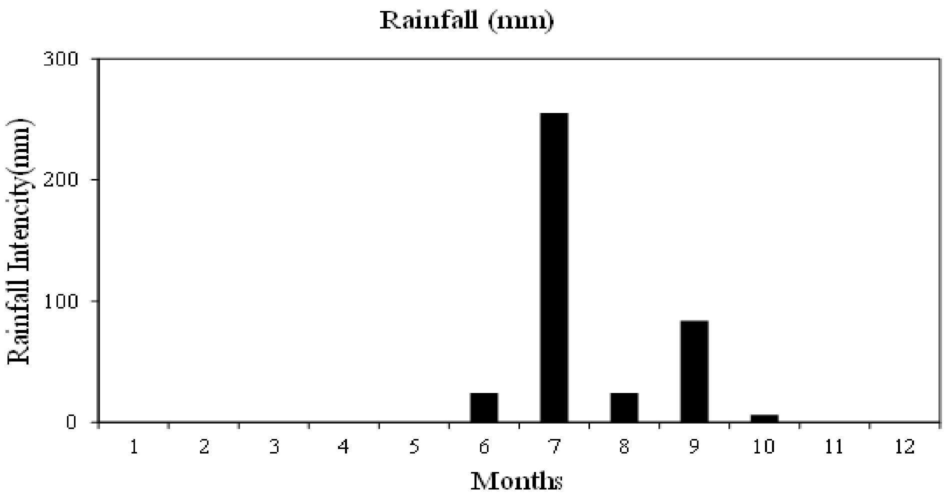


Figure 3: Monthly Rainfall Data in Khulna City (World Weather Information Service, 2012)

It shows that the heavy rainfall only occurs from June to October. In this period adequate rainfall occurs and the surplus rain from this month would be stored to cover the shortage in dry period. Rainwater needs to be stored during the rainy season, so that it could be used throughout the year. The average annual rainfall in Khulna city of 20 year (1988-2007) rainfall data was found 1800 mm/yr. which is adequate for rainwater harvesting to fulfill the drinking water demand of existing population in KUET campus.

3.4 Estimated Demand vs Supply curve

The per capita consumption of drinking water was taken 2 liter/capita/day for the KUET. From Figure 4 it was found that the monthly drinking water demand was found 273 m^3. The monthly potential rainwater supply from average monthly rainfall was calculated and the highest monthly potential rainwater supply was found 3373 m^3. From the demand vs. supply curve it was found that during October to May rainfall is poor and supply is lower than demand. During the month June to September the potential rainwater supply is further greater than drinking water demand and these surplus water could be stored to cover the shortage in dry periods.

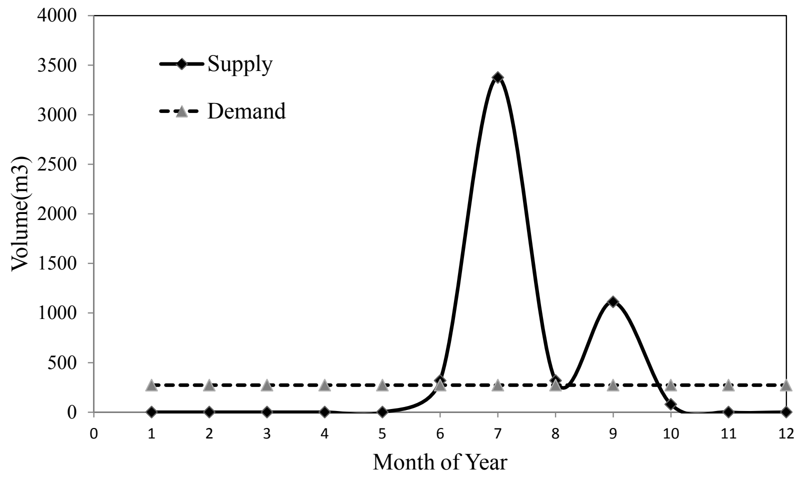


Figure 10: Demand of Drinking Water vs. Supply of Potential Rainwater Using Mass Curve (For KUET Campus)

The per capita consumption of drinking water was taken 2 liter/capita/day for the residential people in Building-14 in KUET campus as a pilot study. From Figure 5 it was found that the monthly drinking water demand was found 1.8 m^3. The monthly potential rainwater suppy from average monthly rainfall was calculated and the highest monthly potemtial rainwater supply was found 50.30 m^3. From the demand vs supply curve it was found that during October to May month rainfalll is poor and supply is lower than demand. During the month June to September the potential rainwater supply is further greater than drinking water demand and these surplus water could be stored to cover the shortage in dry periods.

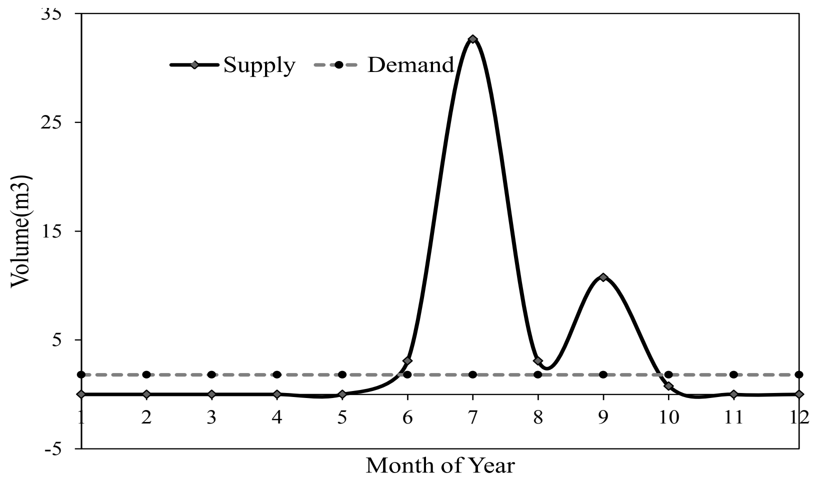


Figure 11: Demand of Drinking Water vs. Supply of Potential Rainwater Using Mass Curve (For Building No-14)

3.5 Cumulative Demand vs. Supply curve

The cumulative monthly drinking water demand for existing population in KUET campus was found to be 3276 m^3 considering the daily per capita demand of 2 liter/capita/day for residential population (Figure 6). The potential rainwater supply was around 5200 m^3. Thus, there would have substantial amount of water surplus after satisfying the existing demand for drinking water. This surplus water we could be stored to supplement the shortage during dry periods.

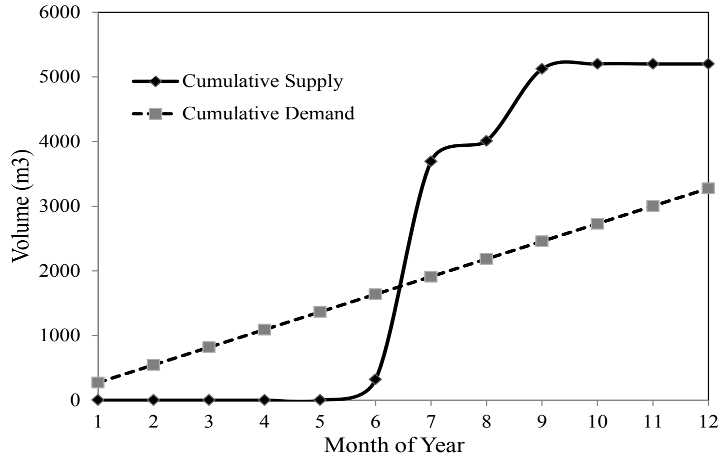


Figure 12: Cumulative Demand vs. Supply of Rainwater Using Mass Curve Method (For KUET Campus)

The cumulative monthly drinking water demand for existing population in B-14 (around 30 persons) was found to be 21.6 m^3 considering the daily per capita demand of 2 liter/capita/day for residential population (Figure 7). The potential rainwater supply was around 50.30 m^3. Thus, there would have substantial amount of water surplus after satisfying the existing demand for drinking water. This surplus water we could be stored to supplement the shortage during dry periods.

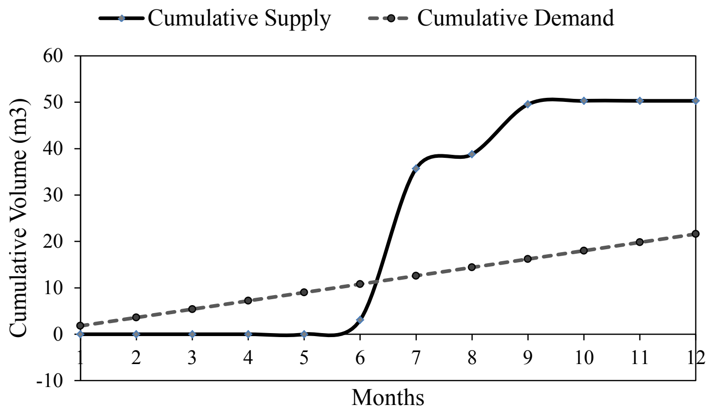


Figure 13: Cumulative Demand vs. Supply of Rainwater Using Mass Curve Method (For Building No 14)

3.6 Proposal for Rainwater Harvesting Technique

Rainwater will be collected from roof top concrete paved areas and then filtered into the sand filter. It should be noted that rainwater harvesting is effective in the summer months by collecting and storing water after a period of rain. In this system rainwater could be harvested directly from the roof top without any energy use. This system of rainwater harvesting by gravity is relatively cost effective with respect to other water supply system (Figure 8).

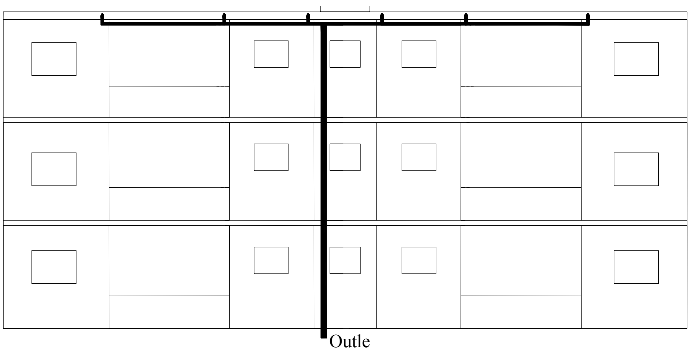


Figure 8: Layout of Pipe System of RWH for Building-14 in KUET Campus

3.7 Storage Tank Installation

The size of the storage tank needed for a particular application is mainly determined by the amount of water available for storage (a function of roof size and local average rainfall), the amount of water likely to be used (a function of occupancy and use purpose) and the projected length of time without rain (drought period). Storage tank with filter (Figure 9 and Figure 10) is installed.

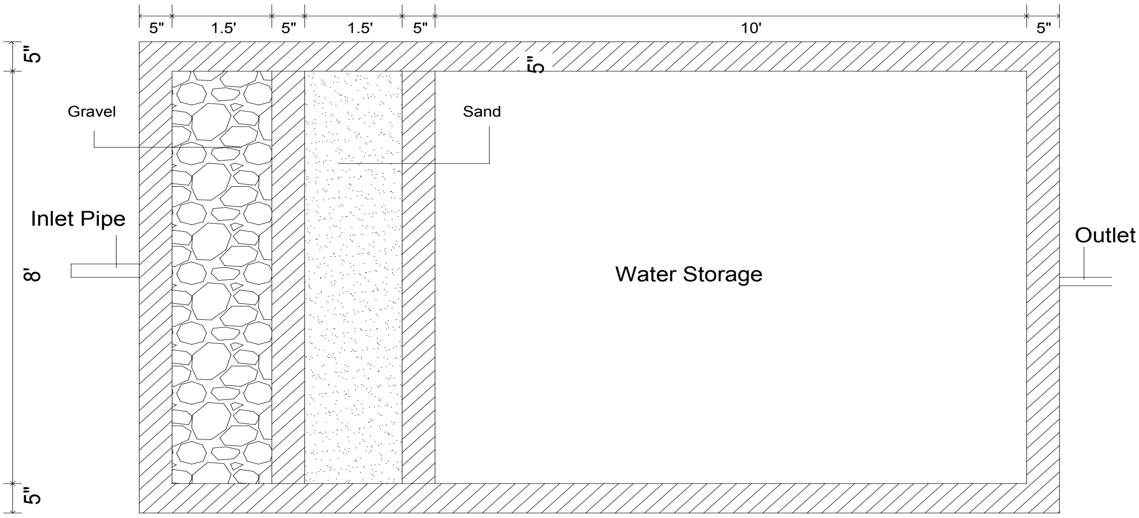


Figure 9: Top View (Plan) of Storage Tank with Filter

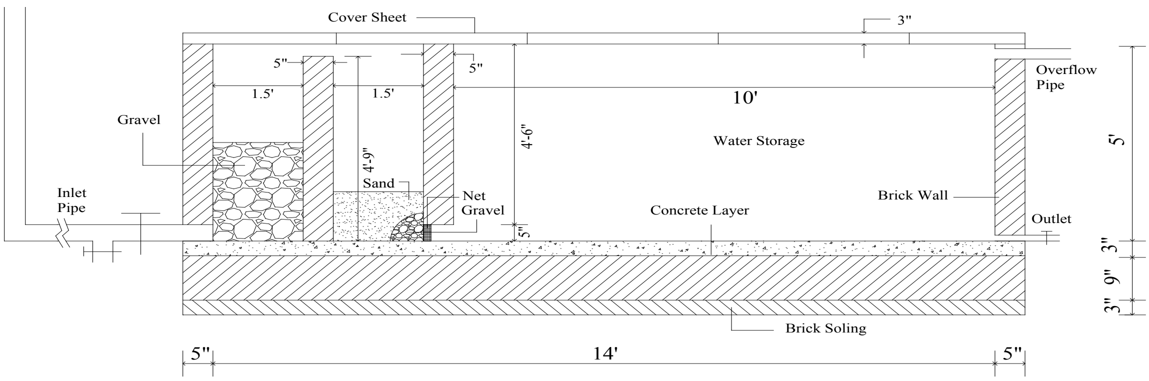


Figure 10: Longitudinal Section of Storage Tank with Filter

In Figure 11, detail specification of filter material (uplift roughing filter followed by sand filter) is shown and size of the filter material is specified.

Specifications for Filter Material:

Gravel 1:10~15 mm dia
Gravel 2: 5~10 mm dia
Sand: Sylhet Sand

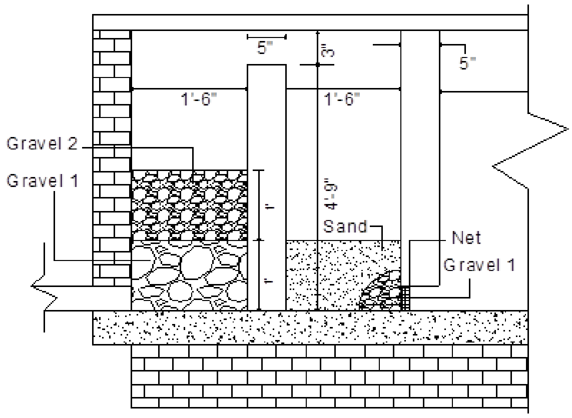


Figure 11: Details of Filter Material

3.8 Cost Analysis

Cost estimated for storage tank has been shown in the following Table 5 and Table 6.

Table 5: Material Cost

SL no.	Materials	Quantity	Unit	Rate(Tk)	Total Cost(Tk)
1	Brick	3800	Nos.	@7500tk for 1000 nos	28550
2	Cement	24	No. of bag	@425tk per bag	10200
3	Sand	120	cft	@25tk per cft	3000
4	10mm~15mm dia gravel	25	cft	@150tk per cft	3750
5	5mm~10mm dia gravel	15	cft	@140tk per cft	2100
6	Sylhet sand	20	cft	@40tk per cft	800
7	Cover Sheet & Piping	135	sft	@10 Tk per sft	3350

Total material Cost = 51750 Tk

Table 6: Labour Cost

SL no.	Work	Quantity	Unit	Rate (Tk)	Total Cost (Tk)
1	Brick Soling	160	sft	@3.00 Tk per sft	480
2	Brick Work	260	cft	@10.00 Tk per cft	2600
3	C.C. work	34	cft	@28.00 Tk per cft	952
4	Plaster with NC	375	sft	@9.50 Tk per sft	3563
5	Gravel Placing	40	cft	@5.00 Tk per cft	200
6	Sand Placing	20	cft	@5.00 Tk per cft	100
7	Cover Sheet setting	135	sft	@2.50 Tk per sft	338

Total Labour Cost = 8233 Tk

Grand Total Cost = Material Cost + Labor Cost = (51750+8233) = **60000 Tk**

4. CONCLUSIONS

It is found that the potential rainwater supply would be accomplished for the drinking water demand during June to September and there have a lake of water during October to May. This surplus water can be stored to supplement the shortage during dry periods. The monthly drinking water demand for forecasted population of KUET campus was 456 m^3. By mass curve analysis, the storage volume requirement was found around 750 m^3 when drinking water consumption was taken 2 liter/capita/day. It was estimated that about 13632 liter of water is needed to be stored for 30 persons considering drinking water demands of 2 liter/capita/day at Bldg-14. A storage tank with gravel-bed roughing filter followed by sand filter was designed for the RWH system. The overall estimated cost was found to be Tk. 60000/= for construction as well as plumbing arrangements.

REFERENCES

Bangladesh Metrological Department (BMD), (2008). www.bmd.gov.bd (Accessed on 9/12/2012)

CIA World Factbook for Bangladesh, http://cia-world-fact-book.findthedata.org, (Accessed on 21/07/2013)

Environment Agency (1999). Saving water: on the right track. A summary of current water conservation initiatives in the UK.

World Weather Information Service, www.worldweatheronline.com, (Accessed on 21/07/2013)

Proceedings of the 2nd International Conference on Civil Engineering for Sustainable Development (ICCESD-2014), 14~16 February 2014, KUET, Khulna, Bangladesh, ISBN: 978-984-33-6373-2 (CD-Rom)

REUSE OF TEXTILE DYING WASTEWATER FOR IRRIGATION IN CULTIVATION OF SOME SELECTED CROPS: A REVIEW

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ABSTRACT

With the industrial development in Bangladesh, the waste management systems did not develop accordingly. Though the textile effluent is a great burden for environmental pollution, it has some useful aspects which may be utilized for beneficial purposes instead of direct disposal. The use of dyeing effluents for irrigation purpose may be an alternative method for reuse which can meet the substantial irrigation requirements in Bangladesh. As per our study, this type of practice in Bangladesh has not yet started extensively. Some researchers conducted their research using different less contaminated wastewater from dyeing process in irrigation of some vegetables where growth, yield and nutritional qualities did not hampared. However, some type of wastewater occured heavy metal contamination in the different parts of vegetables. And some other study was carried out using dilluted wastewater where it was shown that dilluted wastewater is suitable for irrigation. Therefore in this study we tried to make an integrated approach from different research works to assess the reuse of wastewater on some selected crops like Indian Spinach, Steam Amaranth, Pea, Wheat, Tomato etc. This information could be beneficial for practical application of wastewater in irrigation in Bangladesh and can be the sustainable solution to industrial pollution.

Keywords: dyeing effluents, reuse, irrigation, yield and growth, heavy metal, environmental pollution

1. INTRODUCTION

Due to rapid urbanization and industrial development, groundwater resources in most of the areas of the world are shrinking at an alarming rate and may not meet the ever increasing demands from agriculture and industry in future. Estimates revelled that agriculture sector consumes maximum amount of groundwater for irrigation. Irrigation is a key factor in securing food supplies in many developing countries (Pescod, 1992). The disposal of wastewater is a major problem faced by industries, particularly in the case of large industrial areas, with limited space for land based treatment and disposal. As a solution of this problem, in industrialized countries wastewater reuse can be the part of a strategy to protect water bodies and to reduce wastewater treatment costs. Scientists around the globe are working on new ways of conserving water. It is a suitable time, to refocus on one of the ways to recycle water through the reuse of industrial wastewater for irrigation and other purposes. The use of industrial or municipal wastewater in agriculture is a common practice in many parts of the world (Sharma et al., 2007; WHO, 2006). Rough estimates indicate that at least 20 million hectares in 50 countries are irrigated with raw or partially treated wastewater (Hussain et al., 2001) as this effluent contains various micronutrients essential for plant growth.

The major objectives of wastewater irrigation are that it provides a reliable source of water supply to farmers and has the beneficial aspects of adding valuable plant nutrients and organic matter to soil (Liu et al., 2005b; Horswell et al., 2003). The reuse of wastewater for irrigation therefore offers a very promising water conservation alternative. Feachem and Mara (1982) were of the view that the use of wastewater for irrigation often allows the free discharge of wastewater. The disposal of wastewater on land presents not only an appropriate medium for disposal of many wastes but can be a cheaper way of waste treatment. With careful planning and management, the positive aspects of wastewater irrigation can be achieved (WHO 2006). In developed countries where environmental standards are applied, wastewater is treated prior to use for irrigation.

However, in some developing countries where treatment costs cannot yet be afforded, wastewater in its untreated form is widely used in agriculture (Hussain et al., 2001; Friedel et al., 2000**).** Untreated or partially treated wastewater can introduce a huge amount of inorganic and organic contaminates, including heavy metals, into agricultural lands (Wang and Tao, 1998). Irrigation with textile wastewater can change in the basic physicochemical properties of soil (soil quality degradation) is another negative impact of wastewater application in agricultural lands (Chen et al., 2004; Aziz et al., 1999).

As per our study, there is no significant information found about wastewater irrigation in Bangladesh, while different countries are now using wastewater for irrigation after partial treatment or without any treatment. Recently in Bangladesh, some researchers have carried out experimental study in cultivation of some crops by using industrial dyeing wastewater. In this paper, we have tried to integrate an approach to assess the reuse of textile dying effluent in cultivation of some selected crops where Indian Spinach, Steam Amaranth were irrigated by wastewater from different section of a dyeing industry and Pea, Wheat, Tomato were irrigated by diluted wastewater.

2. EEFFECT OF WASTEWATER IRRIGATION ON SOME VEGETABLES COLLECTED FROM DIFFERENT SECTIONS OF DYEING PROCESS.

2.1 Yield and growth of Stem Amaranth and Indian Spinach

In textile dyeing process a huge volume of fresh water and different chemicals are used for colouring and also for other treatments of fabric. However, wastewaters coming from the different steps of dyeing process are not equally polluted and some of them are less contaminated. Therefore, there is a scope to reuse of such type of less contaminated wastewater directly to the irrigation for crops production. Khandaker et al. (2013) shown that less polluted wastewater has no significant effect on yield and growth of Stem Amaranth comparing with groundwater (control) irrigation. They used second wash water after scouring and bleaching (D2), enzyme treated water (D3), second wash after dyeing (D4), second wash after soaping (D5), neutralization (D6), fixing (D7) and also a mixed wastewater (D8) which was collected from equalization tank of effluent treatment plant. Above wastewater were used directly in irrigation of Stem Amaranth. Although, the percentage of yield varied from 14.48 to 51.53 but they were statistically insignificant (Table 1).

Table 1: Effect on growth and yield of Stem Amaranth for textile dyeing wastewater irrigation

Treatments	**No. of leaves per plant**	**Leaf length (cm)**	**Leaf width (cm)**	**Plant Canopy area (cm^2)**	**Stem diameter (mm)**	**Plant height (cm)**	**Plant weight (g) (Yield)**	**% of Yield Variation**
D1	17.70 a	17.53ab	6.79 b	901.30b	8.06a	81.33a	52.14 a	0
D2	17.47 a	16.13 b	6.42 bc	827.30b	6.75a	65.33ab	34.19 a	34.43
D3	18.67 a	16.13 b	6.13 bc	923.30b	7.57a	71.00ab	49.49a	5.08
D4	18.33 a	15.93 b	6.30bc	836.70b	7.96a	58.33ab	27.91a	46.47
D5	19.44 a	16.90 b	7.38ab	902.00b	8.39a	62.33ab	44.59a	14.48
D6	21.60 a	16.60 b	6.43bc	810.00b	8.08a	71.67ab	40.78a	21.78
D7	17.62 a	19.90 a	8.33a	1253.00a	8.06a	66.67ab	36.64a	29.73
D8	14.93 a	11.53 c	5.11c	403.30c	7.30a	52.00b	25.27a	51.53
CV%	19.37	9.95	11.69	17.86	13.78	20.07	47.59	——
LSD value	6.18	2.85	1.35	268.20	1.88	23.23	32.40	——

Note: In a column, means followed by common letters are not significantly different from each other at 5% level of probability by DMRT.

Khandaker et al (2013), conducted an another study on Indian spinach using similar categories of wastewater. In this case, growth indicating parameters of plants did not differ while the yield was to some extent varied comparing with ground water irrigation (Table 2). However, they suggested that these type of less contaminated wastewater could be use in vegetable cultivation where the scarcity of groundwater is high.

Table 2: Effect on growth and yield of Indian Spinach for textile dyeing wastewater irrigation

Treatments	Number of leaves per plant	Leaf length (cm)	Leaf width (cm)	Plant Canopy area (cm^2)	Stem Diameter (mm)	Plant height (cm)	Plant weight (g) (Yield)	% of Yield Variation
D1	19.11a	17.83a	10.78a	654.0a	10.47a	60.34a	81.33a	0
D2	16.50a	13.28ab	8.39ab	740.0a	9.28a	45.28a	60.67ab	25.44
D3	15.55a	12.83b	7.22b	745.0a	10.35a	42.26a	58.00ab	28.68
D4	14.67a	12.00b	7.97b	642.0a	9.80a	40.38a	46.00ab	43.44
D5	15.44a	13.22ab	7.44b	611.30a	10.17a	38.23a	57.33ab	29.50
D6	14.11a	12.22b	6.78b	581.30a	9.16a	35.76a	45.33b	44.26
D7	15.56a	12.78b	7.22b	619.00a	9.13a	51.51a	45.33b	44.66
D8	14.89a	12.56b	7.11b	617.30a	8.60a	45.07a	67.67ab	16.80
CV%	21.48	19.57	19.44	27.36	20.54	39.85	31.65	____
LSD value	5.92	4.57	2.67	312.00	3.46	31.30	31.98	____

Note: In a column, means followed by common letters are not significantly different from each other at 5% level of probability by DMRT.

2.2 Nutritional qualities of Stem Amaranth and Indian Spinach

Khandaker et al. (2013) revealed that there was no significant variation was observed in ascorbic acid (vitamin C) of Stem Amaranth and in Indian Spinach irrigated with different less polluted textile dyeing wastewater comparing with groundwater. Ascorbic acid was varied from 3.52 to 4.54 mg in Stem Amaranth and 4.25 to 5.13 mg in Indian Spinach per 100 g among the different wastewater irrigated plants. The amount of ß carotene in different wastewater irrigated Stem Amaranth varied from 0.49 to 0.82 mg per 100 g and in Indian Spinach it was varied from 0.12 to 0.54 mg per 100g. The amount of chlorophyll a and chlorophyll b of different wastewater treated plants were very close with the amount of groundwater (D1) treated Indian Spinach and Stem Amaranth. The concentration of chlorophyll a and chlorophyll b ranged from 0.14 to 0.32 mgg^{-1} and 0.07 to 0.13 mg g^{-1} in Stem Amaranth and Indian Spinach respectively.

2.3 Heavy Metal Uptake by Stem Amaranth and Indian Spinach

Textile industries use different chemicals depending upon their requirement and purpose of operations and thus generate effluents contaminated with pollutants of different nature and that too in vary concentrations. Residual metal complex dyes in wastewater are the main source of heavy metals. Heavy metals particularly chromium (Cr), copper (Cu), zinc (Zn) and nickel (Ni) are widely used for the production of color pigments of textile dyes (Halimoon and Rachel, 2010). A study was conducted by Khandaker et al. (2013) on accumulation of heavy metal by Stem Amaranth and Indian Spinach for dyeing wastewater irrigation and shown that the concentrations of Cu, Zn, Cr and Ni were below detectable range in groundwater (D1), second wash after bath drop (D2) and in enzyme process (D3) while in other wastewater samples were found to be well below of their respective acceptable limits of irrigation standard (DOE, 2003) (Table 3).

Table 3: Concentration of Heavy Metals in Wastewater

Heavy Metal	Irrigation Standard (mg/kg)	Concentration (mg/kg)							
		D1	D2	D3	D4	D5	D6	D7	D8
Cu	3.0	<MDL	<MDL	<MDL	0.2990	0.6810	0.0050	0.0064	0.1110
Zn	10.0	<MDL	<MDL	<MDL	0.9140	0.1920	0.3160	0.3840	0.3540
Cr	1.0	<MDL	<MDL	<MDL	0.0170	0.0039	0.0326	0.0406	0.0306
Ni	1.0	<MDL	<MDL	<MDL	0.0050	0.0027	0.0229	0.0262	0.0237

Note: MDL- minimum detectable limit of AAS for Cu, Zn, Cr and Ni was 0.002

The uptake of metal ions has been shown to be influenced by the metals species and the different parts of plants Juste and mench (1992). Khandaker et al (2013) studied on accumulation of heavy metals by the different parts (root, stem and leaves) of Stem Amaranth and Indian Spinach was observed by irrigating with some selected less polluted dyeing wastewater. The metals (Cu, Zn, Cr and Ni) investigated in this study were detected in the different wastewater as well as the different parts of these vegetables. The concentration of copper and zinc (2-27ppm and 1-28 ppm respectively) were found below the recommended limits of heavy metals (40 ppm) by FAO (Food and Agriculture Organization) in the both studied vegetables. The amount of Cr concentration in D4 and D8 wastewater irrigated Indian Spinach and Stem Amaranth exceeded the safe limit of health (20 μgg^{-1}) of Indian Standard for Cr in plants. Moreover, nickel was found above the Indian standard (1.5 ppm) in Stem Amaranth (1.52 to 2.2 ppm) and Indian Spinach (1.7 to 2.8 ppm) irrigated with wastewater of enzyme treatment (D3). As growth and yield and nutritional qualities of these two vegetables were not affected significantly for less polluted dyeing wastewater irrigation, therefore, the dyeing wastewater of second wash after scouring and bleaching (D2), neutralization treatment (D5), and second wash after soaping (D6) and fixing treatment (D7) could be reused directly for irrigation purposes of vegetables cultivation to reduce the stress on groundwater.

3. EFFECT OF IRRIGATION ON SOME SELECTED CROPS USING DILUTED TEXTILE DYEING WASTEWATER

3.1 Pea:

Malaviya, (2012) studied of the impact of dyeing industries effluent at five different concentration of effluents (20%, 40%, 60%, 80% and 100%) on growth of pea in pot culture . In his research, dye industry effluent was analyzed for physicochemical characteristics (Table 4) and its impact on growth behaviour of Pea (*Pisum sativum*). Several growth parameters of pea comprised of root and shoot length, root and shoot weight, root-shoot ratio, plant biomass and number of stipules etc. In his study, it was clearly shown that diluted wastewater irrigated plant exposed better results on growth parameters. Plant biomass was the highest (10.78 g) in 20% diluted wastewater irrigated plant while the lowest (3.51 g) biomass was found in raw wastewater (full concentrated) treated plant. Similar growth pattern was found in shoot weight, shoot length, number of stipules etc (Table 5).

Table 4: Physicochemical characteristics of different concentrated diluted dyeing effluent

Effluent (%)	**pH**	**EC (mS/cm)**	**TDS (mgL^{-1})**
20	10.3	0.6	384
40	10.3	1.0	640
60	10.3	1.2	768
80	10.3	1.4	896
100	10.3	1.7	1088

Table 5: Effect of concentrations of effluent on growth parameters of pea after one month of sowing

Efficient treatment (%)	**Root length (cm)**	**Shoot length (cm)**	**Root/ Shoot ratio**	**Root weight (g f. wt.)**	**Shoot weight (g f. wt.)**	**Root/ Shoot ratio**	**Plant biomass (g f. Wt.)**	**Number of Stipules**
Control	8.56 ±0.20	8.44 ±1.53	1.01	2.48 ±0.39	5.62 ±1.44	0.44	8.10	10.72 ±0.52
20	9.38 ±0.53	9.22 ±1.81	1.01	3.90 ±0.43	6.88 ±1.95	0.56	10.78	10.83 ±1.06
40	7.96 ±2.85	7.98 ±0.45	0.99	2.32 ±0.52	4.19 ±0.13	0.55	6.51	10.00 ±0.65
60	7.66 ±1.15	7.78 ±0.26	0.98	2.00 ±1.30	3.95 ±2.39	0.50	5.95	9.11 ±0.66
80	7.34 ±0.28	7.55 ±0.25	0.97	1.60 ±0.32	3.81 ±1.57	0.41	5.41	9.09 ±0.89
100	6.44 ±0.70	6.81 ±0.62	0.94	0.99 ±0.76	2.52 ±1.41	0.39	3.51	8.86 ±0.52

values are mean of three ±SD except for plant biomass

3.2 Wheat

Jolly et al. (2012) Studied on the impact of dyeing industry effluent on soil and crops. The study involved cultivation of wheat on the tub soil through irrigation with the raw effluent as well as treated effluent at different concentration levels (2.5%, 5%, 10 %, 25% and 50%) with groundwater which was collected from a dyeing and finishing industry. This study revealed that wheat plants irrigated with treated effluents shown better performance in growth and yield characteristics compared to untreated effluent irrigated plants. Moreover, the growth and yield performance of wheat plants influenced by the percentage of effluent applied with groundwater (Table 6).

Table 6: Growth and yield of wheat at different concentrations of treated and untreated dyeing effluent

Effluent applied %	**Treatments**	**Plant height (cm)**	**Leaf area (cm^2)**	**Seed dry weight per plant (g)**	**Root dry weight per Plant (g)**	**Number of seeds per plant**	**Seed weight per plant (g)**
0	Groundwater (controlled)	81.65	27.8	15.81	0.557	150	20.96
2.5	Untreated	72.19	26.9	14.47	0.5498	145	18.85
	Treated	83.47	28.2	16.92	.5932	159	21.15
5	Untreated	69.32	25.3	12.35	0.5234	137	16.76
	Treated	86.32	30.60	18.19	0.6423	178	23.31
10	Untreated	68.24	24.6	11.25	0.5184	129	14.72
	Treated	81.29	27.4	13.92	0.5436	149	19.92
25	Untreated	66.12	22.9	10.21	0.5123	124	13.59
	Treated	80.21	26.1	12.86	0.5389	141	17.89
50	Untreated	64.98	21.3	9.20	0.5036	121	11.87
	Treated	77.35	25.2	11.79	0.5130	139	16.76

They also studied that raw effluent as well as treated effluent at different concentration levels which had a significant impact on nutritional qualities of wheat seeds. The percentage of protein and carbohydrate of seeds decreased almost gradually with the increased of the effluent concentrations in irrigation water at the different treatments of wheat plants (Table 7)

Table 7: Nutritional Contents of Wheat Seed Grown on Effluent-Irrigated Soil

Effluent applied %	**Treatments**	**Protein content of seeds (%)**	**Protein content of seeds per Plant (g)**	**Increase/ decrease seed protein content per plant (%)**	**Carbohydrate content of seeds (%)**	**Carbohydrate content seeds per plant (g)**	**Increase/ decrease seed carbohydrate content per plant**
0	Groundwater (controlled)	12.184	2.55	-	68.264	14.31	-
2.5	Untreated	10.484	1.98	-22.35	62.935	11.86	-17.12
	Treated	12.795	2.71	+6.27	69.384	14.67	+2.52
5	Untreated	9.03	1.51	-40.78	57.479	9.63	-32.70
	Treated	13.334	3.11	+21.96	70.032	16.32	+14.05
10	Untreated	8.923	1.31	-48.63	52.854	7.78	-45.63
	Treated	11.049	2.20	-13.72	66.351	13.22	-7.62
25	Untreated	8.521	1.18	-53.73	51.682	7.02	-50.94
	Treated	10.251	1.83	-28.24	63.334	11.33	-20.82
50	Untreated	7.60	0.90	-64.71	45.182	5.36	-62.54
	Treated	9.112	1.53	-40.00	57.174	9.58	-33.05

3.3 Tomato:

Khan et al, (2012) Studied on the effect of textile dyeing wastewater on pot cultivation of tomato plants. For the experimental study, the plants were treated with 20% and 30% effluent concentration and compared with groundwater as control. The plants were harvested at pre-flowering, peak-flowering and post-flowering stages for studying different growth parameters (root and shoot length, dry weight of root and shoot and total dry weight) and also Chlorophyll a, b and total chlorophyll. In their study, it was clearly seen that, in pre, peak and post flowering stages the root length and shoot length of tomato plants were comparatively higher at 20% diluted wastewater treated plant than 30% diluted wastewater treated plants (Table 8a). Moreover, total shoot dry weight of tomato irrigated with 20% diluted water was 32% higher at peak flowering stage that irrigated by 30% diluted wastewater (Table 8b). On the other hand, nutritional qualities also affected for different concentrated wastewater irrigation. The amount of carbohydrate, nitrogen and protein in tomato were 32%, 42% and 43% higher respectively for 20% and 30% concentrated wastewater irrigated plant (Table 8c).

Table 8 (a) : Effect of wastewater on growth and yield of tomato.

Treatment	Root length (cm)			Shoot length (cm)		
	Pre-flowering	Peak-flowering	Post-flowering	Pre-flowering	Peak-flowering	Post-flowering
Control	19.46±1.397	26.28±1.666	32.32±1.785	45.02±2.209	52.88±1.986	68.10±2.559
20%	13.36±1.519 (31.34)	19.30±0.977 (25.56)	20.10±2.906 (37.80)	30.86±0.698 (31.45)	41.50±2.423 (21.52)	52.82±1.289 (22.43)
30%	11.56±1.659 (40.59)	15.62±1.867 (40.56)	15.98±1.492 (50.55)	21.58±1.565 (52.06)	32.92±0.875 (37.74)	40.22±1.947 (40.93)

Table 8(b) Effect of wastewater on growth and yield of tomato

Treatment	Root dry wt. (gm)			Shoot dry wt. (gm)			Total dry wt. (gm)		
	Pre-flowering	Peak-flowering	Post-flowering	Pre-flowering	Peak-flowering	Post-flowering	Pre-flowering	Peak-flowering	Post-flowering
Control	0.534±0.082	0.766±0.091	1.284±0.187	4.078±0.509	4.586±0.464	8.726±0.353	4.612±0.534	5.352±0.543	10.010±0.491
20%	0.284±0.082 (46.81)	0.450±0.063 (41.25)	0.538±0.140 (58.09)	2.716±0.757 (33.39)	2.958±0.281 (35.49)	4.140±0.300 (52.55)	3.000±0.821 (34.95)	3.408±0.302 (36.32)	4.678±0.318 (53.26)
30%	0.196±0.086 (63.29)	0.268±0.080 (65.01)	0.386±0.076 (69.93)	1.878±0520 (53.94)	2.030±0.215 (55.73)	2.406±0.555 (72.42)	2.074±0.605 (55.03)	2.298±0.289 (57.06)	2.792±0.629 (72.10)

Table 8(c): effect of wastewater on food value of tomato

Treatment	Carbohydrate (mg g^{-1})			Nitrogen(%)			Protein (%)		
	Pre-flowering	Peak-flowering	Post-flowering	Pre-flowering	Peak-flowering	Post-flowering	Pre-flowering	Peak-flowering	Post-flowering
Control	571.00±1.581	761.00±1.581	591.40±2.073	1.351±0.006	1.208±0.006	0.628±0.004	8.448±0.041	7.556±0.040	3.930±0.027
20%	432.80±2.588 (24.20)	620.00±1.581 (18.52)	486.60±1.341 (17.72)	0.866±0.005 (35.91)	0.595±0.004 (50.76)	0.307±0.002 (51.04)	5.414±0.002 (35.91)	3.720±0.025 (50.76)	1.924±0.018 (51.04)
30%	395.40±2.073 (30.75)	404.60±1.816 (46.83)	400.60±3.209 (32.26)	0.629±0.005 (53.45)	0.342±0.005 (71.65)	0.229±0.004 (63.56)	3.932±0.031 (53.45)	2.142±0.034 (71.65)	1.432±0.030 (63.56)

4. CONCLUSIONS

Based on literature and reviewed, it can be concluded that, textile wastewater hold a great promise for the improvement of crops growth and yields if proper treatment and management are adopted. The safe utilization of textile wastewater for irrigation to crops requires several precautionary measures. such as selection of appropriate less contaminated wastewater from dyeing process, adequate dilution, selection of crops etc. Beside this, physical properties of soils needs to be reviewed periodically for long term sustainability of the system. Textile dyeing wastewater are little bit rich with nutrients. However, it contains some selected heavy metal. Therefore, before use this type of wastewater needs to know the concentration of heavy metals in wastewater and the levels of heavy metal uptake and its risk for consumption considering health safety. More literature reviewed required on this regarding area to explore the prospective use of textile wastewater in Bangladesh which can reduce the stress on groundwater for irrigation, reduce pollution load of textile dyeing industries as well as can save our environment.

REFERENCES

Aziz, O., Inam, A., and Samiullah, (1999). Utilization of petrochemical industry waste water for agriculture. *Water, Air and Soil Pollution*, 115, 321–335

Chen, Y., Wang, C., Wang, Z., and Huang, S. (2004). Assessment of the contamination and geno toxicity of soil irrigated with wastewater. *Plant and Soil*. 261, 189–196.

Feachem, R., McGarry, M. and Mara, D. (1982). Water, Wastes and Health in Hot climates John Wiley and Sons. U. S. A.

Friedel, J. K., Langer, T., Siebe, C., and Stahr, K. (2000). Effects of long term wastewater irrigation on soil organic matter, soil microbial biomass and its activities in central Mexico. *Biology and Fertility of Soils*. 31, 414–421.

Halimoon, R. and Goh, S.Y. (2010). Removal of Heavy Metals from Textile Wastewater using Zeolite. *Environment Asia*. 3(special issue), 124-130.

Horswell, J., Speir, T. W., and van Schaik, A. P. (2003). Bio-indicators to assess impacts of heavy metals in the land applied sewage sludge. *Soil Biology & Biochemistry.* 35, 1501–1505.

Hussain, I., Raschid, L., Hanjra, M. A., Marikar, F., and Van der Hoek, W. (2001). A framework for analyzing Socioeconomic, health and environmental impacts of Wastewater use in agriculture in developing countries. Working Paper 26. Colombo: International WaterManagement Institute (IWMI).

Jolly, Y.N., Islam, A., and Mustafa,A.I., (2012) .” Impact of Dyeing Industry Effluent on Soil and Crop.”Universal Journal of Environmental Research and Technology. Volume 2, Issue 6: 560-568.

Khandaker, S., Saha,G.C., and Hassan, J,.(2013) “Effect of Textile Wastewater on Yield and Nutritional Qualities of Stem Amaranth”. International Journal of Agricultural Science and Research. Vol.3 (3), 171-8.

Khandaker, S., (2013)” Effects of dyeing wastewater as irrigation water on yield and food value of vegetables” M.Sc thesis on Environmental Engineering, Department of Civil engineering, Dhaka University of Engineering and Technology, Gazipur, Bangladesh.

Khandaker, S., Saha, G.C., and Hassan, J. (2013). “Effect of Textile Wastewater on Yield and Nutritional Qualities of Indian Spinach.”American Journal of Agricultural Science, Engineering and Technology (AJASET), Vol. 2 (6), 1-11.

Khandaker, S., Saha, G.C., and Hassan, J., Bhuiyan¸ A.H., and Kamal, Z. U., (2013) “Accumulation of Heavy Metals at the Different Parts of Vegetables Irrigated with Textile Dyeing Wastewater”. Journal of Applied Phytotechnology in Environmental Sanitation. (Paper Accepted for 2014)

Khan,T.I. and Marwar, R. (2012). “Effect of textile waste water on tomato plant, Lycopersicon esculentum.” J. Environ. Biol. 33, 849-854.

Liu, H., Probst, A., and Liao, B. (2005b). Metal contamination of soils and crops affected by the Chenzhou lead/zinc mine spill (Hunan, China). *Science of the Total Environment,* 339, 153–166.

Malaviya, P., Hali, R. and Sharma, N. (2012).” Impact of dyeing industry effluent on germination and growth of pea (Pisum sativum)”. J. Environ. Biol. 33, 1075-1078

Pescod, M. 1992. Wastewater treatment and use in agriculture. Paper 47. Rome: Food And Agriculture Organization of the UnitedNations.

Sharma, R. K., Agrawal, M. and Marshall, F. M. (2007). Heavy metals contamination of soil and vegetables in suburban areas of Varanasi, India. *Eco-toxicology and Environmental Safety.* 66, 258-266.

Wang, X. J., and Tao, S. (1998). Spatial structures and relations of heavy metal content in wastewater irrigated agricultural soil of Beijing’s Eastern farming regions. *Bulletin of Environmental Contamination and Toxicology*. 61, 261–268.

World Health Organization (WHO). (2006). Guidelines for the safe use of wastewater, excreta and grey water: Wastewater use in agriculture (Volume II). Retrieved from persistent URL: http://www.who.int/water_sanitation_health/

PRESENT SCENARIO OF RAINWATER HARVESTING IN RAJSHAHI

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ABSTRACT

Detail field investigation was conducted to know about the present scenario of rainwater harvesting plants constructed in the arsenic affected areas Charghat and Bagha Upazila of Rajshahi district. Rainwater harvesting system was introduced during the years of 2001 to 2003 by NGO Forum and Somota Nari Kollan Sangstha in the study areas as an alternative source of drinking water. People of the areas used this facility for two to three years out of ten years of design period and stopped using because of facing several problems regarding the aesthetic problem of drinking rainwater and operation and maintenance of this system. Eventually, they have removed the plants by demolishing. The people have lack of awareness regarding harmful effect of arsenic. The economical loss is 50% of the total project cost and service loss is 70% of the total service period due to the demolishing of the rainwater harvesting plant only after three years of use.

Keywords: Rainwater harvesting, arsenic contamination, economic loss, service loss, lack of awareness

1. INTRODUCTION

Rainwater harvesting is a common practice in the countries and areas where the annual precipitation is high and safe drinking water and useable water is insufficient. All over the world, the economical condition has prompted the low-income groups to harvest the rainwater for household and essential uses. People in areas where lack of drinking water, particularly the coastal areas and the rural areas in the country, practice rainwater harvesting (Ferdausi and Bolkland, 2000). The high annual rainfall in Bangladesh makes rainwater harvesting a great logical alternative solution for the arsenic contamination of the groundwater in Bangladesh (Rahman et al., 2003; Boers and Ben-Asher, 1982).

The water supply system in Bangladesh has become extremely dependent on groundwater source for easy and convenient withdrawal of safe water. As a result, installation of the shallow hand-pump tube-wells and deep tube-well has been accelerated for domestic, commercial and industrial water supply. Moreover, excessive groundwater is withdrawing from deep aquifer for irrigation purpose. On the other hand, excessive arsenic in shallow ground water aquifers was detected in the very early nineties. However, the rainwater is free from arsenic contamination and the physical, chemical and bacteriological characteristics of harvested rainwater represent a suitable and acceptable means of potable water (Rahman and Yousuf, 2000). From third century of BC to present time, rainwater harvesting was practiced in many countries including Pakistan, Italy, Srilanka, Australia and India (Lee and Visscher, 1990). Therefore, efforts are being made by the different organizations to develop and promote rainwater harvesting as an alternative safe water supply options for the people in the arsenic affected areas. The immediate challenges therefore, are to assess the various technological options in the terms of their technical feasibility, the economic viability, social acceptability and the environmental sustainability (Choudhury & Vashudevan, 2003).

The presence of arsenic in groundwater in Bangladesh was first discovered in 1993 at Chapai Nawabganj under Rajshahi division. Considering the gravity of the arsenic problem, water samples were tested to identify the groundwater contamination all over the country. It was found that one third shallow tubewell producing arsenic contaminated water (Ahmed and Rahman, 2007). Among the arsenic affected areas in Rajshahi district, Bagha and Charghat upazila are in vulnerable condition for arsenic contamination. Numbers of arsenic patients were found in these areas. As a result arsenic free alternative source of drinking water becomes the prime demand for the people of Bagha and Charghat Upazila. To serve the demand, a lot of rainwater harvesting plant was installed in problem areas of Bagha and Charghat Upazila of Rajshahi District. The aim of this study was to investigate the present situation of rainwater harvesting plant in problem areas of Bagha and Charghat upazila.

2. METHODOLOGY

The information of annual rainfall, rainfall pattern, climatic condition, population, land area, major occupation of the people, the background for the establishment of rainwater harvesting plants were collected. The detail information of the established rainwater harvesting plants such as plant address, type of plant, capacity, catchment area, quality of the harvested rainwater, construction cost, design aspects, etc. were collected from the project implementing organization like NGO Forum and Samata Nari Kallyan Sanstha. A field visit was conducted in study areas to observe the present situations of existing rainwater harvesting plant. Detail questioner survey was carried out among the end users of the rainwater harvesting plants to know about the construction, operation, maintenance, necessity, participation, problems faced by the users, etc.

2.1 Study Area

Bagha and Charghat upazila under Rajshahi district are considered as study areas in this study considering severity of arsenic problem and installation of rainwater harvesting plant. The area is located in between 24º-23´ to 25º-15´ north latitude (Figure 1) and 88º-02´ to 88º-57´ east longitude (Figure 1). The study area includes 13 villages in Bagha and Charghat Upazila namely Miapur, Anupampur, Arazi Sadipur, Chandpur, Talbaria, Kaluhati, Batikamari, Berhabashpur, Fakirpara, Monigram, Jotnasti and Bajubagha. The area of Bagha Upazila is 184.25 km^2 and Charghat Upazila is 164.52 km^2 having about 3290 families in the study area. The average annual rainfall of the study area is around 1400 mm and the highest rainfall occurs in the month of July, which is around 3000 mm. A tropical dry climate with comparatively high temperature prevails in the study area except for the wet season beginning from mid June to early October. The period from November to late February is dry and relatively cool with almost no rainfall in the area. The yearly average temperature ranges from 4ºC to 44ºC and humidity is about 78%.

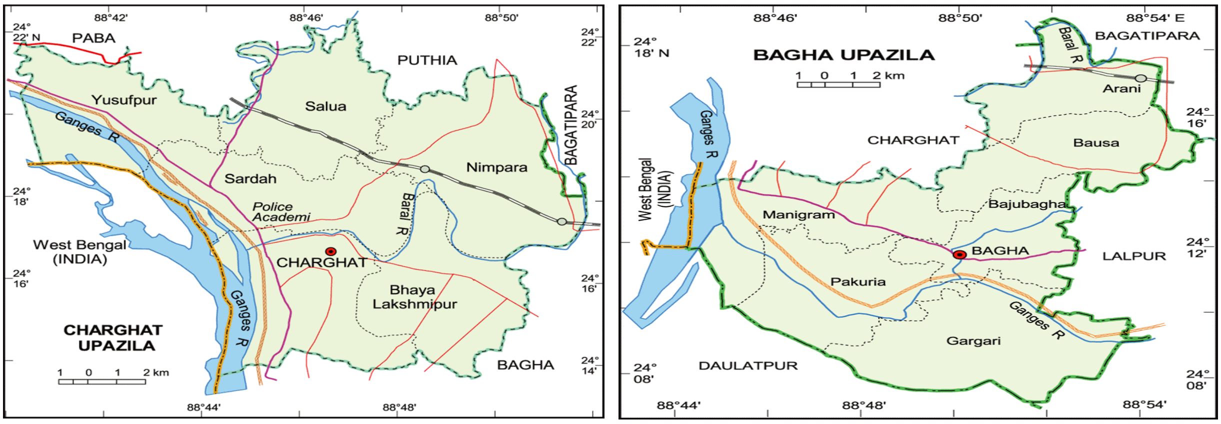

Figure 1: Charghat and Bagha upazila map

2.2 Available Rainfall

The monthly rainfall data from 2007 to 2011 was collected from Meteorological Department and analyzed to know about the rainfall pattern and quantity. The monthly distribution of rainfall in the study area follows the usual pattern of monsoon with heavy rains starting in May and ending in September and very little of no rainfall during the rest of the year (Table 1). The rainfall in the area varies from about 1043 to 1658 mm. The average annual rainfall of the study area is around 1280 mm and the highest average monthly rainfall occurs in the month of September is around 261 mm.

Table 1: Monthly rainfall data of the study area (Bangladesh Meteorological Department)

Month	Rainfall (mm)					
	2007	2008	2009	2010	2011	Average
January	0	0	26	1	0	5.40
February	0	27	0	7	0	6.80
March	7	59	0	28	3	19.40
April	36	13	30	0	200	55.80
May	189	260	144	131	151	175.00
June	188	313	247	126	136	202.00
July	130	364	373	183	183	246.60
August	247	236	245	240	230	239.60
September	302	309	129	282	282	260.80
October	36	76	121	45	52	66.00
November	10	1	0	0	0	2.20
December	0	0	0	0	0	0.00
Total	1145	1658	1315	1043	1237	1279.60

3. FIELD INVESTIGATION

3.1 Rainwater Harvesting Plants in Rajshahi

Rainwater harvesting plants were installed in Charghat and Bagha Upazila by two non-government organizations namely NGO Forum and Samata Nari Kallyan Sangstha (SNKS). They have taken initiatives individually to implement the project for providing rainwater harvesting plant as an alternative drinking water source in these arsenic affected areas during 2001 to 2003. The plants were installed in household and institutional level mainly for domestic and institutional use. The rainwater harvesting plant installation projects were implemented at Dokkhin Monigram, Bagha Girls College and Habashpur under Bagha Upazila and at Miyapur and Kaluhati under Charghat Upazila. Total rainwater harvesting plants of 319 were constructed within study area. The main components of plant were constructed are water collection system and storage tank. Various types of tanks were constructed with different materials and capacities. The summary of the rainwater harvesting plants installed by NGO Forum and Samata Nari Kallyan Sangstha (SNKS) are presented in Table 2 and 3.

Table 2: RWH units installed during the project period by NGO Forum

Sl. No.	Type of Tank	Capacity (Litters)	Size of Catchment, (m^2)	No. of Plants	Unit Cost (Tk.)
1	Ferro-cement Tiles Tank	2500	9-10	4	4500
		3200	10-20	22	6000
2	Ferro-cement Jar	1000	8-9	29	2900
		2000	9-10	8	3500
		2500	6-8	7	4500
3	RCC Ring Tank	1000	6-8	6	2700
		2000	9	1	3800
		2500	9-10	11	4400
4	Brick Tank (SS)	2500	6-8	2	5500
5	Brick Tank (OS)	500	6-8	34	5000
		1000	6-8	5	3500
		2500	9-10	8	1300
6	Chari Tank	650	6	1	1200
		950	7	1	1800
7	Earthen motka	500	6-8	127	550
8	Plastic Tank	500	6-8	2	3300
	Total	25800	-	268	6,50,350

Total 319 rainwater harvesting plants were installed during the period of 2001-2003 with the help of NGO Forum and Somota Nari Kollan Sangstha. Out of these 268 plants were constructed by NGO forum and 51 plants were constructed by SNKS. Both of these NGO's constructed various types of rainwater harvesting system. They used Ferro-cement tank, Ferro-cement jar, RCC ring tank, Sub-surface brick tank, On-surface brick tank, Chari tank, Earthen Motka and plastic tank for water storage. The capacity of these tanks varying from 500 to 3200 litres. The total installation cost varied from Tk 550 to Tk 6000 depending on capacity and types of plants. The service life was considered 10 years with necessary maintenance.

Table 3: RWH units installed during the project period by SNKS

Sl. No.	Type of Tank	Capacity (Litters)	Avg. Size of Catchment, (m^2)	No. of Plant	Unit Cost (Tk.)
1	Brick Tank	500	12	8	3209
		1000	17	5	4016
		2500	22	7	5668
2	Concrete Ring	1000	11	6	3171
3	Ferro-cement Jar	1000	20	2	3243
		2500	21	8	4427
4	Ferro-cement Tank	2500	27	14	4657
5	Earthen Motka	500	12	1	3834
	Total	11500	142	51	2,15,388

3.2 Quality of Harvested Rainwater

Quality of harvested rainwater was evaluated by NGO Forum at regular interval taking samples from different types of storage tanks during the project period. The water quality parameters including pH, Turbidity, Total Coliform, Fecal Coliform, Iron, Zinc, Calcium, Magnesium, Fluoride and Lead were examined. The test results of these important water quality parameters of the study area tested by NGO Forum are given in Table 4. The test results show that total coliform and fecal coliform are not found in storage water. However, pH, Turbidity, Iron, Calcium, Magnesium, Lead, Zinc and Fluoride were traced. All samples show alkaline and most of the cases pH level is higher than allowable limit.

Table 4: The quality of harvested rainwater (NGO Forum)

Sl. No	Water quality parameters									
	pH	TC	FC	Fe (mg/L)	Ca (mg/L)	Mg (mg/L)	Pb (mg/L)	Zn (mg/L)	Turbidity (NTU)	Fluoride (mg/L)
1.	8.45	0	0	< 0.05	36	90	0.0110	0.1075	9.1	< 0.05
2.	9.06	0	0	< 0.05	39	39	0.0123	0.3725	9.2	< 0.05
3.	11.02	0	0	< 0.05	4	38	0.0138	0.0630	0.5	< 0.05
4.	7.32	0	0	< 0.05	27	45	0.0109	0.1056	0.4	< 0.05
5.	8.79	0	0	< 0.05	40	22	0.0103	0.1816	5.2	< 0.05

3.3 Present Scenerio of RWH Plants

During the period of 2001-2003, total 319 rainwater harvesting plants were installed in the study area by NGO Forum and Somota Nari Kollan Sangstha. The rainwater harvesting plants were used by the people of the study areas for almost three years after the construction. In the field visit, it was observed that most of the plants have been demolished by the users from their own premises. At present, a very few number of rainwater harvesting plants are found in some houses in the study area. The detail information of the existing plants is given in Table 5.

Table 5: Current condition of existing rainwater harvesting plants

Location	Capacity (Litters)	Type of Tank	Catchment area, (m^2)	Implementing Organization	Present Condition
Habashpur, Bagha	2500	RCC Ring Tank	10	NGO Forum	Inactive
Kaluhati, Charghat	2000	Ferro-cement Jar	9	NGO Forum	Inactive
Dokkhin Monigram, Bagha	2500	Ferro-cement Jar	21	Somota Nari Kollan Sangstha	Inactive
Bagha Girls, College	2500	Brick Tank	22	Somota Nari Kollan Sangstha	Inactive
Miyapur, Charghat	500	Earthen Motka	12	Somota Nari Kollan Sangstha	Inactive

The rainwater storage tank used in the study eareas are presented in Figure 2, 3 and 4. Figure 2 shows that storage tank made of concrete ring with capacity of 1000 liters. Presently, this is not in use. Ferro-cement jar type water storage tank is shown in Figure 3 with the capacity of 2500 liters which is also not in use. An earthen motka is shown in Figure 4 that supose to be used for storing rainwater but used for keeping crop.

Figure 2: Concrete ring water storage tank at Habaspur

Figure 3: Rainwater harvesting system at Dhokkhin Monigram

Figure 4: Motka used for keeping dry crops

4. FINDINGS

During the field investigation, it was found that not a single RWH plant is in using condition. Only a few house woner/plant user kept the water reservoir in their premises. Most of the plants were demolished already by the user and they started to drink tube well water. A questioner survey was conducted in Charghat and Bagha Upazila among the caretakers of rainwater harvesting plants. Twenty two and twenty seven house owners/caretakers were interviewed in Charghat and Bagha Upazila, respectively. Questions were asked about the reasons of installation of RWH plant, awareness of arsenic problem, mode of participation, satisfactory level, present condition of plant, reasons of not using plant presently, reasons of demolition of tank, etc.

4.1 The Opinions of Plant Users

1. Several users tolled that they did not know about the effect of arsenic on health. The presence of arsenic in their tube-well water was also unknown. However, they found some people had been suffering for skin disease for long time and some of the cases were severe.
2. The NGO workers tested the tube-well water and informed them about the presence of arsenic in their groundwater. The local people also could know about the effect of arsenic and the causes of skin disease of the affected people of their locality.
3. The NGO workers informed them to drink arsenic free water and encourage for drinking rainwater. Moreover, they expressed their intention to install rainwater harvesting plant in each household having arsenic contaminated groundwater.
4. There were no contribution in cost sharing in installation but house owners had to take responsibility to operate the plant.
5. The people used the plants for two to three years after installation without satisfaction. They have mentioned several reasons of dissatisfaction in rainwater drinking. These can be mentioned in the following points:
   a. People are habituated to use tube-well water for drinking for long time.
   b. Taste of rainwater is not so pleasant like groundwater.
   c. Most of the roofs used as catchment are under the three and causing dirty easily and frequently.
   d. Cleaning of roof before raining is problem for people especially at sudden occurrence of rainfall.
   e. Facing difficulty in operating the plant and collecting the clean water mainly due to the absence of caretaker during the rain.
   f. Cleaning of storage tank is also discouraging people to use the RWH plant.
   g. Bad smell and odor were found in the rainwater for storing in the tank for a long time.
   h. Various types of worms/insects were found in the stored rainwater tank which creates the aesthetic problem for drinking.
   i. People didn't get interest to use rainwater for drinking purposes due to insufficient rainfall.
   j. Many people thought that the tube-well water become arsenic free due to not using for long time and again they have started to drink groundwater collecting from tube-well.

6. All the rainwater harvesting plants are no longer using and most of the plants have been demolished because of several reasons, such as:
   a. The people are not feeling the necessity of drinking rainwater.
   b. The drinking of rainwater has forced them to change their drinking habit.
   c. The people have started again to drink groundwater collecting with tube-well and the rainwater harvesting plant become useless.
   d. The rainwater storage tank occupied a large space within the small house of village people. So they demolished the tank to remove the unnecessary obstruction.
7. Some people have started to collect drinking water initially from green-marked arsenic free tube-well of neighbor house to avoid drinking rainwater. Sometimes this collection of drinking water from neighbor bothers the green-marked arsenic free tube-well owners and they forbidden the collector to take water from their tube-well. This sort of situation and the prestige issue insisting the people to return to drink arsenic affected water from their own tube-well.
8. The monitoring of the plants was stopped by the NGOs at the end of project implementation. The NGOs did not have any responsibilities to keep the project operational and make the system sustainable.
9. Users and project implementing organization were accusing each other for the present situation of the rainwater harvesting plants. Some users argued that the plants were used for two to three years and failed to run successfully for the long run because most of the people are of low income level and a little literate. They do not have the ability to keep the plant operational with proper maintenance. Therefore, project implementing organization should have continuous support because they are getting fund from donor agencies.
10. On the other hand, project implementing organizations are telling that they have some limitations to provide continuous support. They have got fund for some specific activities like awareness build up, project implementation and monitoring of water quality for the particular period of time. At the end of the project period, definitely users have to take responsibility to make it sustainable.

4.2 Economic Loss of The RWH Plant

Total economic loss of the rainwater harvesting project in Charghat and Bagha Upazila was estimated considering the total project cost, operation and maintenance cost, design period and utilization period. The project supposed to serve for 10 years i.e., design period was 10 years. The total capital cost for constructing the 319 rainwater harvesting plants was Tk.865738.00 (Tk.650350.00 + Tk.215388.00). Yearly operation and maintenance cost was considered to be 10% of total capital cost. Salvage value and discount rate (i) were considered to be 1% and 10% of capital cost, respectively. Therefore, annual cost of the project can be estimated by using the following equation.

Annual cost = Capital cost × CRF + Annual O & M cost – Salvage value × SF ---------------------- -----(1)

Here, Capital Recovery Factor (CRF) = $\{i\,(1+i)^n\} \div \{(1+i)^n - 1\}$ --- (2)

$= \{(1+0.1)^{10} \times 0.1\} \div \{(1+0.1)^{10} - 1\} = 0.163$

Sinking Fund (SF) = $1 \div \{(1+i)^n - 1\} = 0.627$ -- (3)

Annual cost = (865738 × 0.163) + 8657.38 – (8657.38 × 0.627) = Tk.144344.50

Total cost for 3 years = (144344.50 × 3) = Tk.433033.50

Total loss of the project = (865738 – 433033.50) = Tk.432704.50

Percentage of loss = {(432704.50 × 100) ÷ 865738} = 50%

The rainwater harvesting project was designed for 10 years. But the project runs for 3 years only. Initially the total capital cost was Tk.865738. The total loss of the project is Tk.432704.50. The percentage of loss is 50% in terms of money. However, the project loss is 70% in terms of service period. It is meaning that remaining 70% service could be received with only 50% of the project cost.

5. CONCLUSIONS

Charghat and Bagha Upazila of Rajshahi district are highly arsenic affected areas. The rainwater harvesting plants were constructed at these areas during the period of 2001-2003. Two NGOs namely NGO Forum and Somota Nari Kollan Somity (SNKS) constructed the plants by the help of foreign aid. People of this region used all the rainwater harvesting plants for almost three years after construction and stopped to use for several reasons. Presently, the rainwater harvesting plants became useless to them and demolished the system. People are now using tube-well water for drinking purpose instead of rainwater though they are not sure about the safety of water. It is understand from field investigation that the project was not based on demand driven rather supply driven and most of the people are not aware about the harmful effect of arsenic on health. As a result, the project was sustained for only three years and failed with 50% economic loss and 70% service loss.

REFERENCES

Ahmed, M.F. and Rahman, M. (2007).Water Supply and Sanitation – Rural and Low Income Urban Communities, ITN-Bangladesh.

Boers, Th.M. and Ben-Asher, J. (1982). A review on rainwater harvesting, Agricultural water management, 5(2), 145-158.

Choudhury, I. and Vasudevan, L. (2003). Factor of biological contamination of harvested rainwater for residential consumption. Department of Construction Science, Texas A&M University, College Station, Texas 77843. http://www.watercache.com/docs/rwquality1.pdf

Ferdausi, S.A. and Bolkland, M.W. (2000). Rainwater harvesting for application in rural Bangladesh, Proceedings of 26th WEDC Conference, Dhaka, Bangladesh, 2000. Edited By John Pickford. ISBN: 9780906055892

Lee, M.D. and Vissher, J.T. (1990). Rainwater harvesting in five African countries. Occasional paper series, No. 14. www.scribd.com/doc/57190860/Roof-Water-Harvesting

NGO Forum, Rajshahi, Personal communication.

Rahman, M.H, Rahman, M.M, Watanabe, and Yamamoto (2003). Arsenic contamination of ground water in Bangladesh and its remedial measures. In: Arsenic contamination in groundwater-Technical and Policy Dimensions.

Rahman, M.M. and Yusuf, F.M.S. (2000). Rainwater harvesting and the reliability concept. 8th ASCE Specialty Conference on Probabilistic Mechanics and Structural Reliability, PMC2000-084. http://www3.nd.edu/~pmc2000/pmc2000/sessions/papers/p084.pdf

Somota Nari Kollan Somity, Rajshahi, Personal communication.

EFFECTS OF SALINITY ON LAND FERTILITY IN COASTAL AREAS OF BANGLADESH

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ABSTRACT

Salinity intrusion is an increasing problem in the coastal areas of Bangladesh. Climate change and its associated hazards like cyclone, sea level rise, and storm surge have been increasing the salinity problem in many folds. The coastal region covers about 20% of the country, from which cultivable lands are more than 30%. Agricultural land uses in these areas are very poor, because of high content of salinity. Already, 830,000 million hectares of land have been identified which are affected by soil salinity at different degrees. Salinity causes unfavorable environment and hydrological situation that restrict the normal crop production throughout the year. Saline soils contain soluble salts in quantities that affect plant growth adversely, the lower limit for a saline soil being set conventionally at an electrical conductivity of 4 mmho/cm in the soil saturation extract. Nutrient deficiency of N is quite dominant in saline soils. Deficiencies of micronutrients, such as Cu and Zn are widespread. Ions that contribute to soil salinity include Cl^-, SO_4^{2-}, HCO_3^-, Na^+, Ca^{2+}, Mg^{2+}, and, rarely, NO_3^- or K^+. Salinization affects the metabolism of the organisms present in the soil, drastically reducing soil fertility and increasing water proofing of the deeper layers.

Keywords: Climate change, coastal areas of Bangladesh, cultivable land, salinity, micronutrients.

1. INTRODUCTION

"The wealth of a nation lies in her soils and their intelligent development", said a philosopher, Richard Gordon Moores. There are historical evidences that survival of a civilization depends on soil productivity. Soil can be singled out as one of the most important environmental factors affecting crop yields. In many parts of the world, coastal areas are highly populated and often the most developed stretches of land. It is estimated that 40% of the world population lives within 100 km of a coastline and expectations are that this figure grows further in the coming half century. In Bangladesh, about 36 million people live in the coastal areas and livelihood primarily depends on agriculture, fishery, forestry etc. During the natural calamities tidal flooding through a network of tidal creeks and drainage channels connected to the main river system inundates the soil and impregnates them with soluble salts thereby rendering both the top and subsoil saline (Haque, 2006). Salinity causes unfavorable environment and hydrological situation that restrict normal crop production throughout the year. The freshly placed alluviums from upstream in the coastal areas of Bangladesh become saline as it comes in contact with the sea water and continues to be inundated during high tides and ingress of sea water through creeks. Even though salinity intrusion is a slow process but the impact is devastating. National Adaptation Programme of Action (NAPA) Bangladesh 2005 warned that the impact of saline water ingression in estuary and underground water is likely to be accelerated by land subsidence, sea level rise, and low flow river condition. World Bank (2000) predicted a 1 m sea level rise at the end of the century which would affect 17.5% of total land mass of the country. It implies that the future sea level rise will bring further land under inundation and therefore, salinity will intrude to more inlands. Soil salinity is believed to be mainly responsible for low land use as well as cropping intensity in the area (Rahman & Ahsan, 2001). The main obstacle to intensification of crop production in the coastal areas is seasonally high content of salts in the root zone of the soil. Due to drainage congestion, the area remains waterlogged, increasing the salinity (Abedin, 2010).

Bauder (2007) recommended the ideal pH range for irrigation water as 6.5-8.4 and warned that pH above 8.5 might speed up the corrosion of irrigation system. pH level increases due to higher concentration of carbonate and bicarbonate ions and make the irrigation water alkaline. Higher carbonates and bi-carbonates in irrigation water create insoluble minerals with calcium and magnesium ions leaving sodium ions as the dominant ions in the solution. Higher alkalinity in irrigation water reduces micronutrient availability. Higher pH containing irrigation water when applied to land is likely to increase the soil pH as well as sodium level at the same time decline the rate of water infiltration and drainage by blocking the pores with lime and other dispersed particle (Gale *et al.*, 2001). Salt affects crop germination and density, as vegetative development, reducing productivity and, in the most serious cases, leading to plants death, limiting nutrient absorption and reducing the quality of the existing water. For example, elevated salinity weakens plants due to the increase in osmotic pressure and the toxic effect of the salts. In an indirect way, soil salinization can abruptly affect plant growth, due to destruction of the soil configuration and its consequent compacting. This occurs due to a dispersion of the clay particles caused by substitution of the calcium (Ca^{2+}) and magnesium (Mg^{2+}) ions present in the complex by sodium (Na^{+}), resulting in an increase in soil solidity, which is, in the percentage of exchangeable sodium (PES), that, in the last instance, is the main factor responsible for the deterioration of the physical properties of salt-affected soils. The excessive amounts of salts provided by irrigation waters can have adverse effects on the chemical and physical properties of the soils and on their biological processes (Garcia & Hernandez, 1996; Rietz & Haynes, 2003; Tejada &Gonzalez, 2005). These effects include mineralization of the carbon and nitrogen and the enzymatic activity, which is crucial for the decomposition of organic matter and liberation of the nutrients necessary for sustainability of the production (Wong *et al.*, 2008). In addition, the agricultural practices can increase or reduce the microbial population, thus altering the activity, source and persistence of the enzymes in the soil.

The microbial communities of the soil perform a fundamental role in cycling nutrients, in the volume of organic matter in the soil and in maintaining plants productivity. So it is important to understand the microbial response to environmental stress, such as high concentrations of heavy metals of salts, water and the fire content of the soil. Stress can be detrimental for sensitive microorganisms and decrease the activity of surviving cells, due to the metabolic load imposed by the need for stress tolerance mechanisms (Chowdhury, 2011). In a dry hot climate, the low humidity and soil salinity are the most stressful factors for the soil microbial flora, and frequently occur simultaneously. Saline stress can increase importance, especially in agricultural soils where the high salinity may be a result of irrigation practices and the application of chemical manures. Research has been carried out on naturally saline soils, and the detrimental influence of salinity on the microbial communities of soil and their activities reported in the majority of studies (Rietz & Haynes, 2003; Sardinha *et al.*, 2003). Increase in salinity intrusion and increase in soil salinity will have serious bad impacts on agriculture. The food production does not seem to have a better future in the event of a climate change. In recent future, rice production may fall down by 10% and wheat by 30% by 2050 (IPCC, 2007). The aim of this study was to exploring the existing salinity level in the south west coastal soils of Bangladesh together with assessing and quantifying its impact on crop production. Finally, proposals were made to address the issues related to salinity ingression.

2. METHODOLOGY

The saline water intrusion plays a harmful role for agricultural land, the micro-organisms cannot leads their life because of high saline with high content of N, P. Though the fertility of lands is low, the salinity ingression leads more vulnerable condition to the land. Due to the climate change, the monsoon comes fast and a regular flooding occurs at the coastal region. Though most of the lands are on the floodplain, so it is easy to intrude saline water and water logging in parts of the basin areas in the dry season. At recent future, this problem will be an unimpeachable threat to our economy.

2.1 Study Area

The study was conducted at the coastal zone of Khulna region and it is located at the south-western part of Bangladesh (Figure1). The population in Khulna region is 2378971. The economic activities of the population in this zone are fishing, agriculture, shrimp culture. Due to topographical feature, the lands of study area are low lying and about 70% of lands are on flood plain basin. So due to flood or other natural disasters these areas are subjected to flooding and water logging in parts of the basin areas in the dry season. Tidal flooding through a network of tidal creeks and drainage channels connected to the main river system submerges the soil and impregnates them with soluble salts thereby rendering both the top and subsoil saline. Although the lands fertility is low in study area, saline water intrusion may leads them more hazardous. The amount of production of crops decreases abruptly because the blockage of pore in sub-surface of lands due to saline water intrusion and thereby loss of yield.

The study employed by both qualitative and quantitative approaches to collect and analyze data. Both the primary and secondary sources were considered to collect the required data. Key informants interview, case study, household interview, personal observation. The secondary information has been collected through assessing a number of scientific and policy study. The review of secondary literature helps to identify the level of salinity in previous years in different coastal district in Bangladesh and what impacts it have laid on fertility of agricultural land. The data collection for the study has been framed according to the following structure in (Figure 2)

Figure 1: Study area

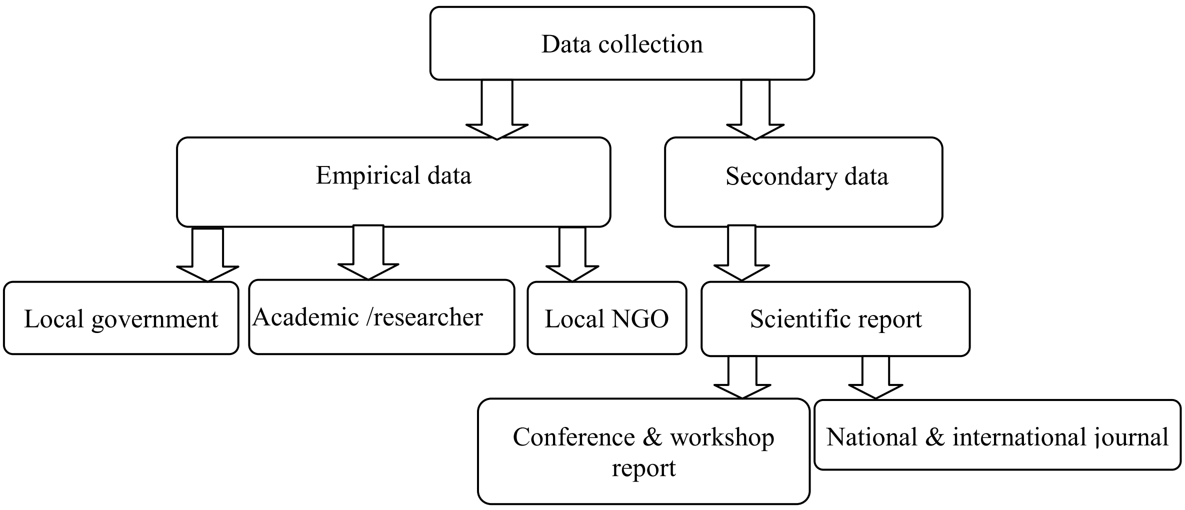


Figure 2: Method of data collection from study area

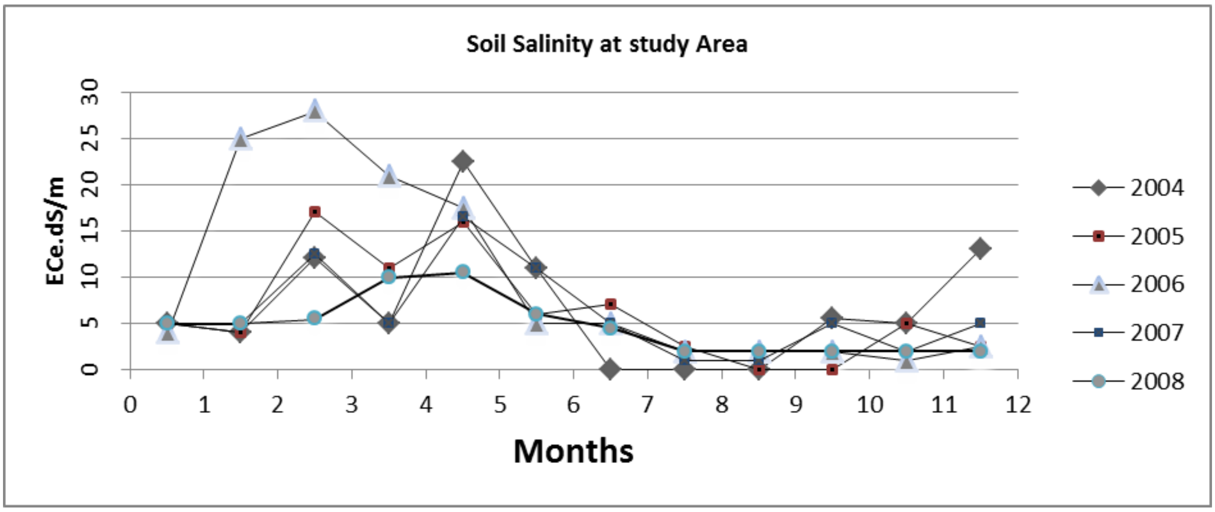
3. RESULTS AND DISCUSSION

The amounts of saline in the soil of study area are found in variable quantity. The organic matter (OM) in the study area was found 0.1-0.3%. Where the limiting values of organic matter is ranges from less than 1% to 1.5%. Low organic content in soils indicates poor physical condition of the coastal soils. The soils contain variable levels of exchangeable bases, but a exceptional feature is the higher Na and Mg saturation of the exchange complex compared to Ca and K in most of the soils. The content of Na and Mg saturation of the exchange complex is harmful because they destroy the soil physical properties and offset plant nutrition. Magnesium has compatible effect of plant uptake of Na as well as incompatible effect on the uptake of Ca and K. The total N contents of the soils are generally low which ranges from 0.1-0.3%. The low N content may be attributed to low organic matter contents of most of the soils in study area. Available P status of the soils ranges from 8-36 ppm. Widespread Zn and Cu deficiencies have been observed in the study area.

Table 1: Chemical characteristics of soils in Khulna

District	pH	OM %	Total N%	CEC %	Na %	K %	Ca %	Mg %
Khulna	6.7-8.5	0.1-0.3	0.1-0.3	18.2-40.6	1.6-33.3	0.3-1.0	8.3-22.5	2.6-18.3

Since the soil organic matter, and consequently, the biomass and microbial activity, are generally more relevant in the first few centimeters at the surface of the soil, salinization close to the surface can significantly affect a series of microbiologically mediated processes. This is a significant problem, since the microbial processes of the soil control its ecological functions and fertility. Due to salinization, electrical conductivity of the soil increases and the microbial community cannot survive in those soils. From Figure 3, it is seen that from 2004-2008 the conductivity of soil in Batiaghata upazilla has given variable values due to salinity. The month of March at 2006, the electrical conductivity was found a maximum value (28). The Electrical Conductivity varies with the month in that year. Also the values were found variable quantity in other years.



(Source: Ahsan and Sattar, 2010)

Figure 3: Salinity effects at different years in study area

The most pressing problem faced by the farmers in the study area is yield loss. However, the yield loss is not uniform for all food crops. Different types of crops respond to salinity differently even at a same level. In the study area the farmers face a great problem during the cultivation, while using the saline water. Table 2 indicates the yield loss due to salinity for different types of crops. The Aus and Amon are the main crops among of them. In accordance to the farmers, due to using saline water for several years, the salt has accumulated in the soil for several years. Also there has inadequate flushing during wet season. After the seeding, it becomes red color after

6-7 days of irrigation. Plants becomes week, also it turns into reddish color and finally get burnt. If saline water is used for irrigation during ripening of rice, insects attacks the paddy. That causes yield loss a great extent. Also in case of onion, the size of the plants become shorter, weak, and also the root got rotten. Chili becomes dried before harvest time. Table 2 shows the gradual reductions of yield. Where at previous years, the production was much better than recent years.

Table 2: Reduction of Yield in study area

Crop	Yield		
	Past(Before 3-4 years) (ton/ha)	Present (ton/ha)	% Reduction
Boro	5.5-6.5	2.5-3.5	50
Aus/Aman	4-5	2-2.5	50
Onion	4-6.5	3-4.5	71
Chili	2.5-2.7	0.6-0.9	75
Jute	3.5-4.7	1.5-2	45

Salinity intrusion in river water may cause economic loss in terms of crop yield reduction. The agriculture production is likely to decrease as saline containing water reduces plant growth through concentrating salt in the root zone of plant and resulting in nutrients imbalance and yield loss. However, salinity in river water has diversified impact on socioeconomic factors, the study deals only with the impact of salinity on agriculture in the study area. Therefore, the study intends to explore the present level of salinity in the irrigation water and its associated hazards in the crop production.

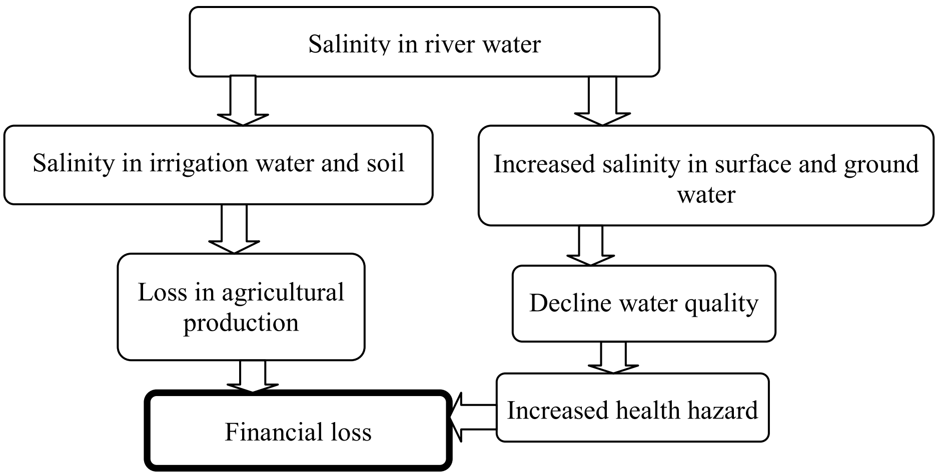


Figure 4: Economic loss of country

3.1 Management of Saline Soils

The principal objective of the recovery of soils affected by salts is to reduce the concentration of soluble salts and of exchangeable sodium in the soil profile, to a level that does not prejudice the development of crops. A decrease in the degree of salinity involves the process of dissolution and consequent removal by percolation water, whereas a decrease in the exchangeable sodium content involves its displacement from the exchange complex by calcium before the leaching process. The reports are in the literature that the efficiency of washing the mocker zone was higher when irrigation was carried out by dripping rather than by other methods. The main question in the recovery of soils affected by salts using irrigation by dripping, is that a reasonable irrigation regime must be carried out to guarantee not only normal growth of the crops, but also an excess of water has to leach out the salts. In case of high concentration of salinity in root zone, deep-water irrigation is required. If soil salinity is too high comparing to the desired level of a certain vegetable in its root zone and the root is 12 inch depth, 6 inch of water is capable of leaching salinity up to 50% and 12 inch of water is capable of leaching salinity up to 70% and 24 inch by approximately 80%. Irrigation method and adequate drainage are also influential aspects of managing soil salinity. Simple surface water run-off cannot leach soil salinity rather water should be drained through soil. In this regard, sprinkler (dripping) irrigation provides a better control of water application rates. The salinity removal diagram shows that after applying the process for three years continuously, the saline would remove from land. Also the pH level can be reduced by iron sulphate and calcium sulphate. Proper drainage facilities should keep in the land to flow out the water. If the drainage system is zigzag type the water cannot stagnant in a place or cannot move fast. Other management techniques can be taken like short time plants irrigation, selection of saline tolerant plants. Rain water using for irrigation is also a good technique. So the water storage tank should available.

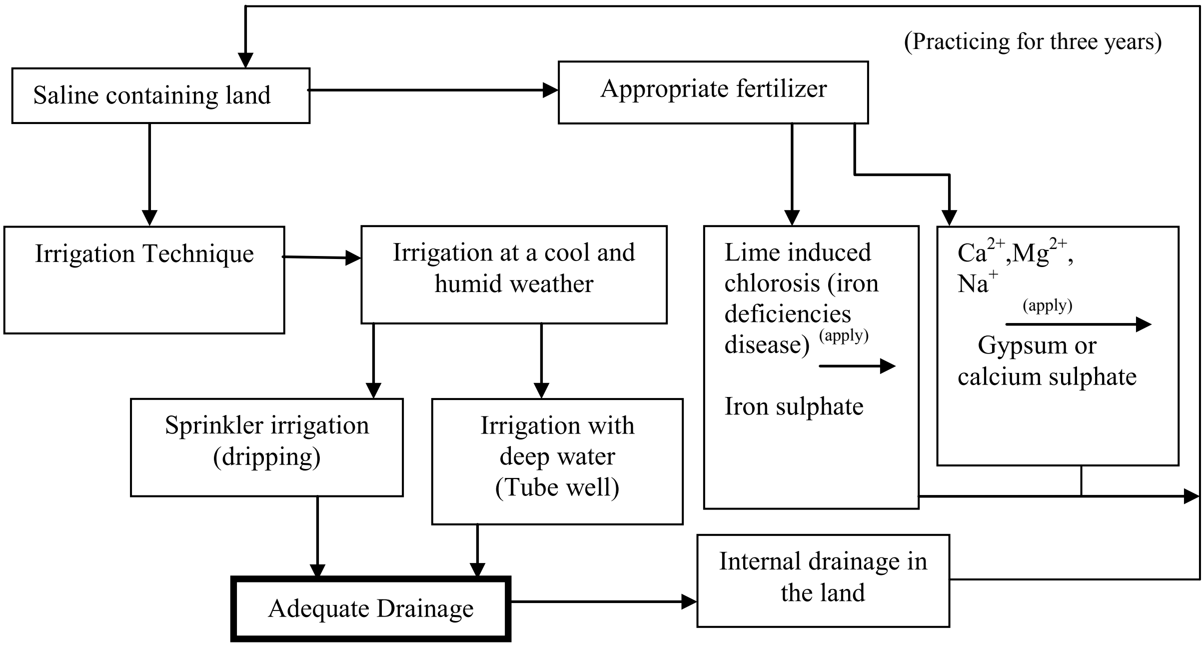


Figure 5: Proposed flow diagram for saline removal from land

4. CONCLUSIONS

Soil salinity is a great problem in the Khulna division. Due to climate changes, seasonal crops are cultivated wrong time. The robi crop faces a great problem due to this change. The effect of saline water on *boro* and *aus* crops is greatly. Due to annual flooding in this area saltwater intrude into land and increase pH and vanishes micro-organism and thus loss the fertility. This may hamper to our total economy. Thus combating land salinization problem is vital for food security in the country through adoption of long-term land management strategy. The remedial measures which have proposed above can slowly recover the problem of saline from cultivable land. The Government of Bangladesh should help and facilitate the landless, marginal and small farmers through group based approaches, especially poor and vulnerable groups through agricultural input support and micro-financing grant in farming practices and non-farm income generating activities. Strengthening the interdisciplinary efforts involves governmental and non-governmental agencies to agriculture land use zoning. Identify and introduce salinity tolerant crop and vegetable varieties for the local people. Salinity is a vital hazard which cannot be ignored for our environmental as well as economic prosperity of our country. For better management practice focus should be given on remission of the salinity in the southwestern coastal area of Bangladesh.

REFERENCES

Abedin, M.Z. (2010). Global Warming, Climate Change And Impact on Agriculture in Bangladesh. 4th National Convention of Krishibid Institute, Bangladesh.

Ahsan, M. and Sattar, M.A.(2010). Coastal areas and saline soils of Bangladesh: their extent, salinity status, management practices and future research needs. Paper presented in the Workshop on Soil Fertility, Fertilizer Management and Future research Strategy, held at BARC, during (18-19 January, 2010).

Bauder,T.A., Waskom R. M., and Davis, J.G.(2007). Irrigation water quality criteria, Crop Series Irrigation. http://cospl.coalliance.org/fez/eserv/co:6315/ucsu2062205062007internet.pdf (Access date 13/07/2013).

Chowdhury, N.; Marschner, P. & Burns, R.G. (2011). Soil microbial activity and community composition: impact of changes in matric and osmotic potential. Siol Biology and Biochemistry, Vol. 43, No 6, (June, 2011), pp. 1229-1236, ISSN: 0038-0717.

Gale, J., Koenig, R., Barnhill, J., (2001). MANAGING SOIL pH IN UTAH. Utah State University, No. AG-SO- 07 http://extension.usu.edu/files/publications/publication/AG-SO-07.pdf (Access date 28/8/2013).

Garcıa, C. & Hernandez, T. (1996). Influence of salinity on the biological and biochemical activity of a calciorthird soil. Plant and Soil, Vol. 178, No. 2, (1996), pp. 255–263,ISSN: 0032-079X.

Haque, S.A. (2006). Salinity Problem and Crop Production in Costal Regions of Bangladesh.Pak. J. Bot. **38**(5): 1359-1365.

IPCC. (2007). Climate change (2007). Synthesis Report . Inter governmental Panel on Climate Change. National Adaption Programme of Action (NAPA).Dhaka, Bangladesh.

Rahman, M.M. and M. Ahsan. (2001). Salinity constraints and agricultural productivity in coastal saline area of Bangladesh, Soil Resources in Bangladesh: Assessment and Utilization.

Rietz, D.N., Haynes, R.J. (2003). Effects of irrigation induced salinity and sodicity on soil microbial activity.Soil Biology & Biochemistry, Vol. 35, No 6, (June, 2003), pp. 845-854, ISSN 0038-0717.

Sardinha, M.; Muller, T.; Schmeisky, H. & Joergensen, R.G. (2003). Microbial performance in soils along a salinity gradient under acidic conditions. Applied Soil Ecology, Vol. 23, No 3, (July, 2003), pp. 237–244,

Schimel, J.P. Balser, T.C. & Wallenstein, M. (2007). Microbial stress response physiology and its implications for ecosystem function. Ecology, Vol. 88, No 6, (June, 2007), pp.1386-1394.

Tejada, M. & Gonzalez, J.L. (2005). Beet vinasse applied to wheat under dry land conditions affects soil properties and yield. European Journal of Agronomy, Vol. 23, No 4, (December, 2005), pp. 336-347,

Azam, F. & Ifzal, M. (2006). Microbial populations immobilizing NH4+-N and NO3--N Differ in their sensitivity to sodium chloride salinity in soil. Soil Biology & Biochemistry, Vol. 38, No 8, pp. 2491– 2494.

Wong, V.N.L.; Dalal, R.C. & Greene, R. S. B. (2008). Salinity and sodicity effects onrespiration and microbial biomass of soil. Biology and Fertility of Soils, Vol. 44, No 7, (August, 2008), pp. 943-953.

World Bank (2000). Bangladesh: Climate Change & Sustainable Development. Report No.21104 BD, Dhaka. http://wwwwds.worldbank.org/external/default/WDSContentServer/WDSP/IB/2001/04/13/000094946_01033105302920/Rendered/PDF/multi0page.pdf (Access date 20/08/2013).

CLIMATE INDUCED VULNERABILITIES IN NATUN BAZAR CHAR AREAS: WATER SUPPLY AND SANITATION PERSPECTIVE

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ABSTRACT

Nowadays climate changes are an important issue in the world. The world has begun to witness the consequences of climate change with the increased frequency of Tsunami, cyclones, and devastating floods which are seriously affecting its helpless populace and leaving them in limitless miseries. Khulna, the third-largest city in Bangladesh, is located in the southwest of the country, where the consequences of climate changes are expected to be particularly severe because of its geographical location and low elevation (Four meter above the sea level). The possible increase in precipitation due to climate change coupled with sea level rise will make the situation even worse. In this study, present water supply and sanitation (WSS) situation in Natun Bazar Char areas have been investigated. To assess the climate induced vulnerabilities on water supply and sanitation system in the Char area, a total number of 50 tubewells and 50 latrines with the most vulnerable condition have been selected. The climate induced rise in water level in the nearby Rupsha River was forecasted and its impact on WSS was predicted. This study revealed that 78% tube wells will be inundated and 58% latrines will be water logged and lost their usability by 2050.

Keywords: Climate Change, Geographical location, Sanitation, Water Level Rise, Water Supply

1. INTRODUCTION

Climate includes patterns of temperature precipitation, humidity, wind and seasons. "Climate change" affects more than just a change in the weather; it refers to seasonal changes over a long period of time. The global climate change is one of the most significant environmental issues of the present world (Chowdhury and Debsharma). The world has begun to witness the consequences of climate change with the increased frequency of Tsunami, cyclones, and devastating floods which are seriously affecting its helpless populace and leaving them in limitless miseries. The effects of global climate change are evident now, as we are experiencing through irregular weather conditions, these effects are multidimensional and the effects are not the same across the globe, because of geographic locations and different levels of development. In recent years, climate change related on water supply and sanitation impacts have also taken precedence. According to Intergovernmental Panel on Climate Change, global warming would cause pollutes the water source thus increase of vector borne and water borne disease in the tropics (IPCC, 2001and 2007). Due to climate change, sea level, humidity, rainfall and temperature is changed. Bangladesh is a low-lying deltaic country in South Asia formed by the Ganges (Padma), the Brahmaputra (Jamuna) and the Meghna rivers and their respective tributaries. The country has been suffering from various types of major natural disasters like floods, cyclone, storm surge, tidal bore, river bank erosion, salinity intrusion and drought etc. Currently climate change poses a new threat to life and livelihood of the people of Bangladesh. Climate change is recognized as a key sustainable development issue for Bangladesh (WB,2000). These risks will be additional to the challenges the country already faces. Long-term changes in temperature and precipitation may impact on water supply and sanitation. Sea level rise may have severe implications on water supply and sanitation of coastal area through inundation and salinity. It has also various impacts on Bangladesh, a coastal country with a 710 km long coast on the Bay of Bengal. A sea level rise of 1m will inundate 17.5% of the country's vast coastal area and flood plain zone. The possible increase in precipitation due to climate change coupled with sea level rise will make the situation even worse to rivers. Temperature rise in the atmosphere causes sea level rise (SRL) and affects low lying coastal areas and deltas of the world. The Natun Bazar Char area under Khulna district has been experiencing frequent water logging during rainy season. The sanitation facilities become unusable during floods and rainy season. This study includes an outlook of remedial measurements that can be considered for sustaining the water supply and sanitation due to climate change in Natun Bazar Char areas. The main objectives of this study is to analyze the climate induced vulnerabilities on water supply and sanitation (WSS) facilities in Natun Bazar Char areas using primary and secondary data with their projection with regards to climate changes.

2. METHODOLOGY

This study has been carried out in one zones representing high vulnerability in Khulna district with an aim to explore and find the effect of climate change on water supply and sanitation in the study area. At first, preliminary field survey was conducted and tube wells (50 nos) and latrines (50 nos) are selected in such a way that it represents the entire study area of Natun Bazar Char. After finalizing the points final field survey was done to collect the Reduced Level (RL) of the selected tubewells and latrines. In the meantime a detail questionnaire survey was conducted to investigate the present water supply and sanitation situation. To assess the climate induced vulnerabilities on water supply and sanitation additional secondary data was also collected from Water Development Board, KCC and various journal and scientific research paper. Finally, a Linear regression model was developed by considering the high water level of Rupshs River to observe the future water supply and sanitation condition in Natun Bazar Char area.

3. RESULTS AND DISCUSSION

3.1 Baseline Water Supply and Sanitation Situation in Natun Bazar Char Areas

It is very likely that climate change has an impact on the quality, quantity and availability of water in the study area. However, climate change is only one of several drivers impacting the water sector in the study area. It cannot always be seen as an isolated factor but in many aspects as an exacerbating factor for other drivers, for instance when the pollution generated by the discharge of untreated wastewater combines with reduced Rupsha river flows due to droughts. In practice, impacts are compounded by the vulnerability of water supply and sanitations (WSS) services, exacerbated by the poor status of the infrastructure in the study area and of the organisations which operate them. It is observed from the questionnaire surveys that water supply situation in the study areas are not satisfactory. Moreover, the other services like sanitation, drainage and waste disposal are not adequate and needs improvement in most of the low cost areas and slums. The overall pictorial view of water supply and sanitation situation in the study area are shown in Fig 1

Figure 14: Pictorial view of present water supply and sanitation situation in the study area.

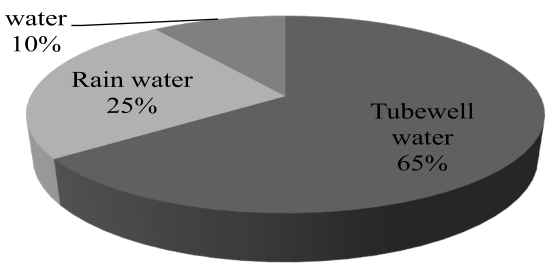

Figure 2: Source of water for drinking and other purposes

The main source of drinking water is the tubewell supply water. Fig.2 shows that around 65%, 25% and 10% water are used for drinking and other purposes from tube wells, rain water and river water, respectively in the study area. But most of the tube wells are salinity affected and base reduced level of these tube wells are very low which may be inundated in near future due to sea level rise. Two other drinking water sources are rain water and river water. But rain water collection system is not hygienic. Therefore, due to increased unavailability of fresh water the people are forced to drink contaminated water. As a result people are affected by different water borne diseases like diarrhea, cholera, etc. People in general have poor knowledge about sanitation in Natun Bazar char areas. Fig.3 Shows that most of the people use pit latrine (70%), but the people are not aware about the use and maintenance of their latrine. During the field survey, it was observed that the pit latrines are not properly lined and covered and due to the leakage of septic tank, waste water comes out the surrounding environment and creates the odor problem and pollutes the surface water. The cleaning system of these latrines is too much worse because cleaning wastes are stored in the open place or thrown in the drain. It is observed that most of the people especially male and children urinate in the open space or near the drain. During the field survey, it was also observed that the sanitation situation becomes worse during the rainy season due to high water level and wastewater contamination in the surrounding areas. Lack of sufficient awareness and people's negligence for cleanness, hygienic condition of most of the latrines is very disappointing and fouling and odors problem are very common.

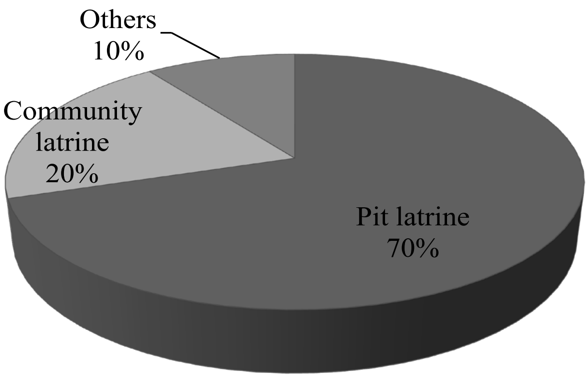

Figure 3: Sanitation practice in Natun Bazar Char areas

3.2 3.2 Scenario of Main Climate Factors for the Study Area

Annual rainfall and temperatures

The total annual rainfall in the study area is changing significantly. Figure 5 shows the year wise mean annual rainfall in the study area. The data was collected from Bangladesh Agriculture Bureau Council (BARC). Previous data for last 60 years (1948-2008) shows that maximum monthly average precipitation 233.25mm and minimum monthly average precipitation 71.83 mm occurred in year 1974 and 1971, respectively (Fig.5). This shift in seasonal rainfall patterns together with increasing intensity and frequency of extreme events is likely to result in flooding on the one hand and scarcity of fresh drinking water on the other. With the prospect of summer droughts, more frequent extreme rainfall and increased flooding, water management is becoming a serious challenge for the study area. As a result fresh drinking water will be in increasingly short supply in the study area.

Over the last 60 years the year 1969 was the hottest, when the average annual air temperature was recorded at 32.02 ºC. The coldest year was 1981, when the average annual air temperature was 28.89 ºC (Fig. 4). The observation records of the past 60 years show the average annual air temperatures varying between 29.89°C to 32.02°C. However, precipitation and temperature of this region is varying frequently, but due to greenhouse effect temperature of the world increasing rapidly. For this more water is vaporing from the water source and increasing the water demand.

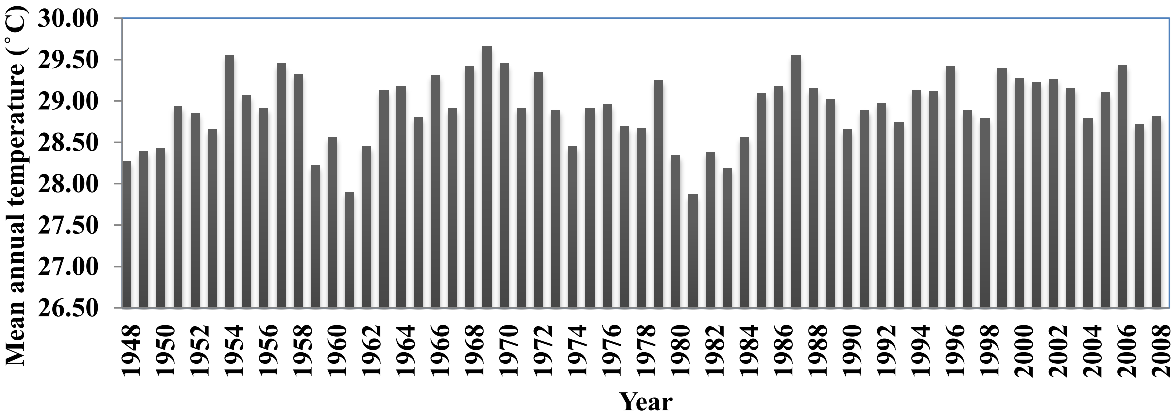

Figure 4: Variation of mean annual temperature of previous year in the study area (BARC)

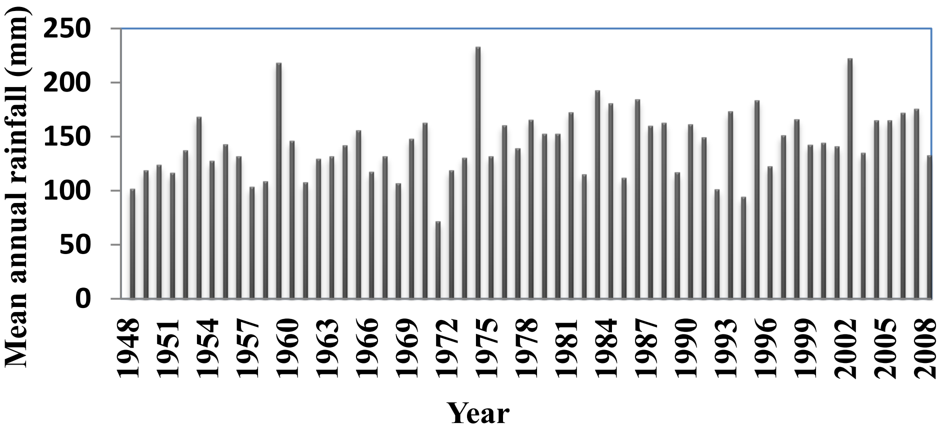

Figure 5: Variation of mean annual rainfall of previous year in the study area (BARC)

3.3 Climate Induced Sea Level Rise in the Study Area

Projected Future Sea Level Rise (SLR) in the Bay of Bengal is provided by IPCC which is shown in Fig.6. From this figure it is found that sea level will rise 14cm and 32cm within the year 2030 and 2050, respectively according to A2 (high emission scenario). This data is used in this report because sea level rise in the Bay of Bengal will be coupled with the high water level of Rupsha river. As a result the study area will be inundated. The main impact of sea level rise on water resources is the reduction of fresh water available due to salinity i

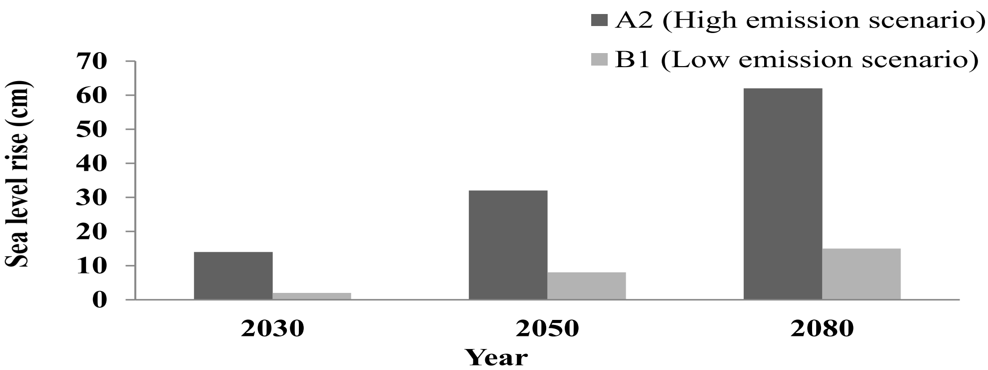

Figure 6: Projected future sea level rise scenario (IPCC, 2007)

3.4 Forecasting of Water Level Rise of Rupsha River

As the river Rupsha is situated near the study area its water level variation will affect the water supply and sanitation system. A linear correlation is made for forecasting of future high water level of Rupsha river by using the previous year high water level of Rupsha River. Fig .7 shows the high water level of Rupsha River from 1978 to 2009. It is seen from the figure that the water level increases gradually from 1978 to 2009 and can be expressed by a linear regression equation. The potential high water level (HWL) in Rupsha River was forecasted Fig. 8 using the equation (1). The predicted rise in water level of Rupsha River by the year 2020 and 2050 were found to be 3.6 m and 4.45 m, respectively. Note that the SLR is added with the value found from the equation (1). $y = 0.022x - 40.97$ (1)

Where, y = high water level (m)
x = corresponding year

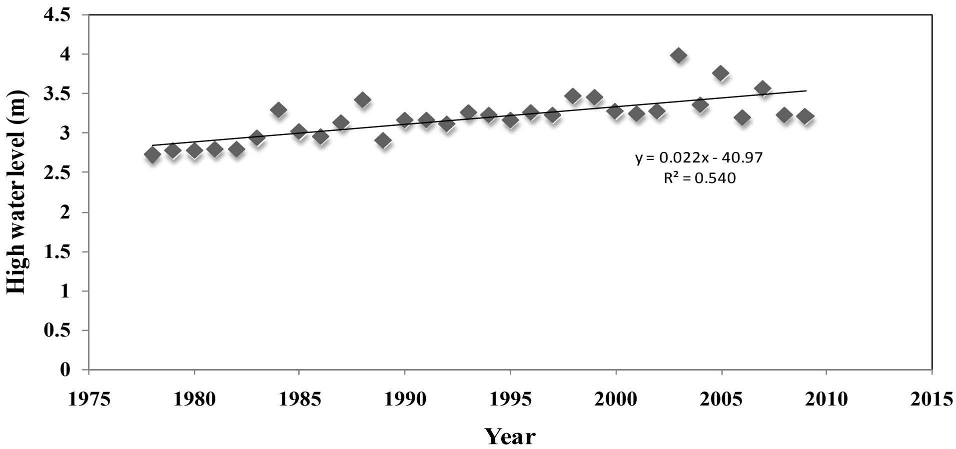

Figure 7: Linear regression model for forecasting of high water level of Rupsha River

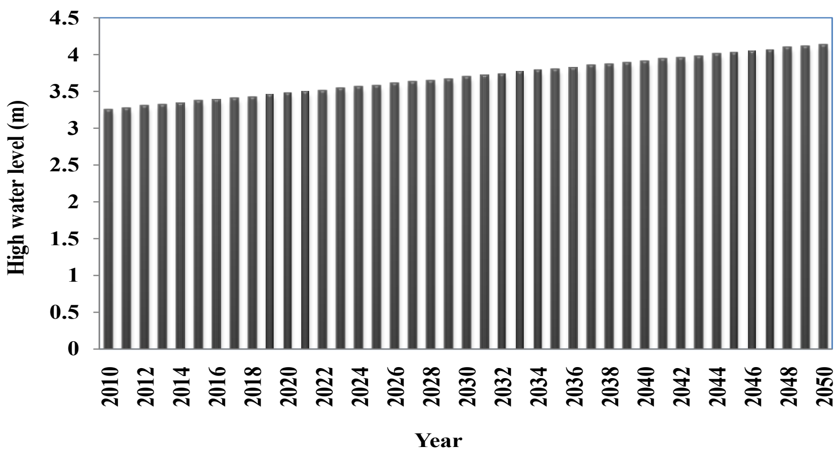

Figure 8: Future high water level of Rupsha River

To investigate how many tubewells and latrines would not be useable by the year 2020 and 2050, the reduced level (RL) of each tubewell platform and latrine base slab was determined from field survey in the study area and then compared with the high water level including sea level rise.

Fig. 9 and 11 show the RL of tubewells and latrines. Red color straight lines represent the HWL of Rupsha River including sea level rise. Tubewells and latrines having base RL below HWL, will be flooded. Fig.10 represents that 7 nos and 39 nos of tubewells will be inundated by the year 2020 and 2050, respectively, which is 14% and 78% of total tubewells examined and Fig.12 represents that 5 nos and 29 nos of latrines will be water logged and lost their usability by 2020 and 2050, respectively which implies 10% and 58% of total latrines.

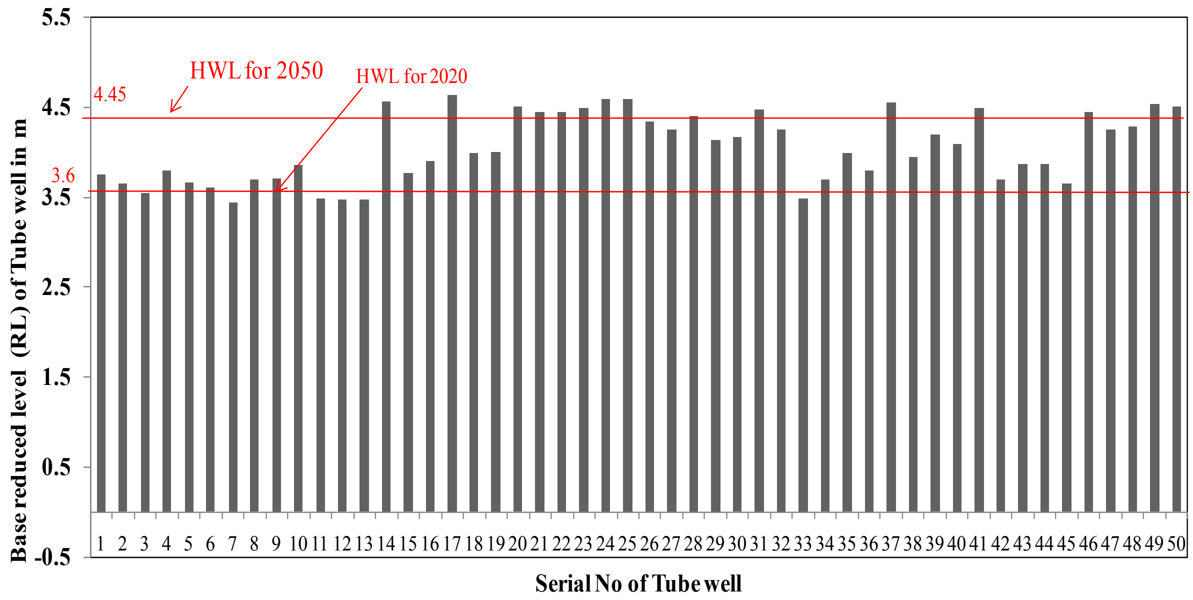

Figure 9: Base reduced level (RL) of selected tube wells.

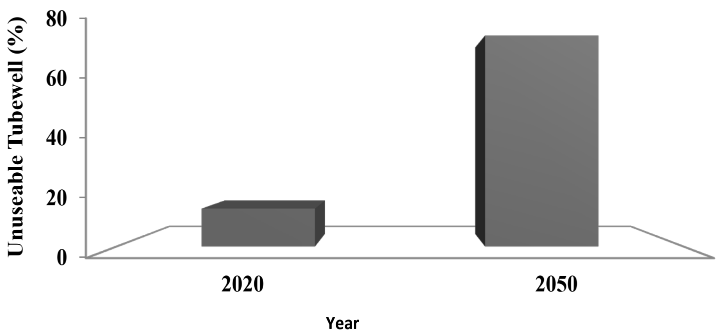

Figure 10: Future predicted unusable tubewells in Natun Bazar Char area by 2020 and 2050.

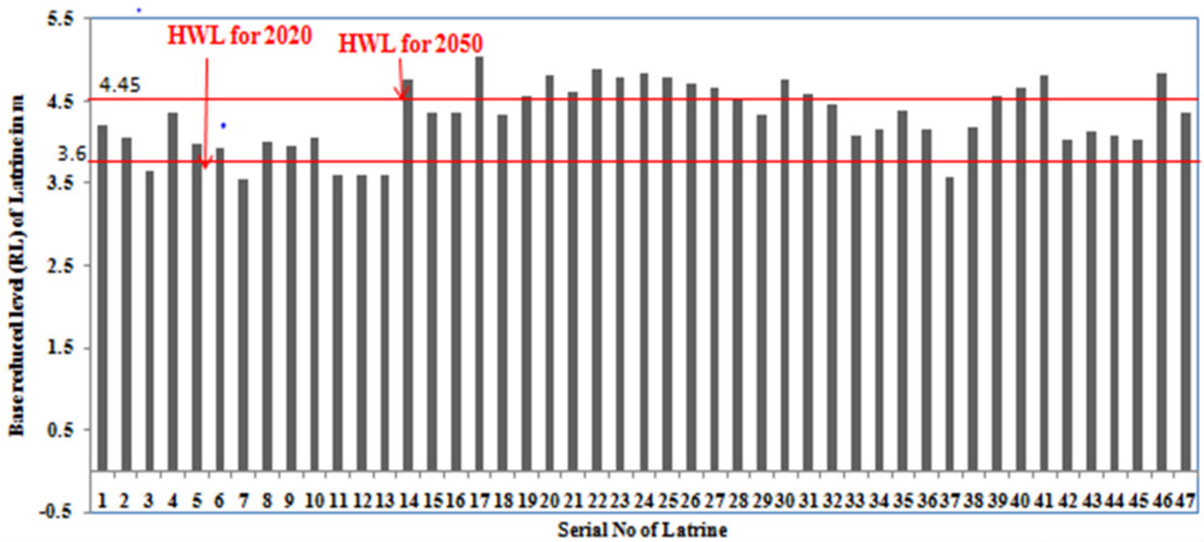

Figure 11: Base reduced level (RL) of selected latrines

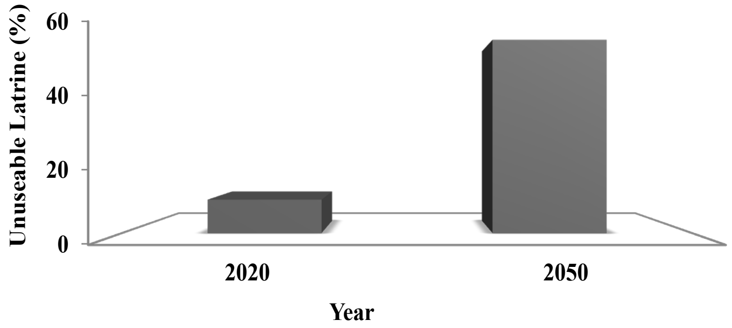

Figure 12: Future predicted unusable latrines in Natun Bazar Char area by 2020 and 2050.

4. CONCLUSIONS

The water supply and sanitation facilities in Natun Bazaar Char area were found extremely vulnerable and become worsen day by day due to climate changes. In most cases, tubewells are used in this area for drinking and other domestic purposes. However, most of the tubewells are affected with high salinity and the base platform level is very low which would be inundated in near future due to sea level rise. In general, local people have poor knowledge about sanitation in Char areas. Most of the people use pit latrines (around 70%); but they are not aware of the proper use and maintenance of these latrines. It is not wise to think that the sea level will not rise, or to wait and see what will happen in the future. Instead, development and implementation of adaptation policies and appropriate mitigation strategies must be identified to respond to the issue of sea level rise. Base of the tube wells and latrines should make sufficient high. Research is needed to find practical solutions to the potential problems and to develop knowledge and capability in order to facilitate improved future water resources management plans. Adequate coastal protection must be developed including drainage facilities. Research is also required to ascertain how to save the country's coastal people threatened by the sea level rise. Technical and financial support from the international community is necessary to combat the impact of sea level rise in Bangladesh. Bangladesh alone is not able to face such a large scale problem.

ACKNOWLEDGEMENTS

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REFERENCES

Chowdhury, M.H.K. and Debsharma, S.K. (1992). "Climate change in Bangladesh-A statistical review". Report on IOC-UNEP Workshop on Impacts of Sea Level Rise due to Global Warming, NOAMI, held during 16-19 November 1992, Bangladesh.

Bangladesh Water development Board, Government of the people's Republic of Bangladesh.

BARC (Bangladesh Agricultural Research Council), Ministry of Agriculture, Government of the people's Republic of Bangladesh.

IPCC (Intergovernmental Panel on Climate Change), (2001) Working Group I Third Assessment Report, Cambridge University Press. Cambridge, UK. p. 881

IPCC (Intergovernmental Panel on Climate Change), (2007) Impacts, Adaptation, and Vulnerability, Contribution of Working Group II to the Fourth Assessment Report, Cambridge University Press, Cambridge, UK. p. 1000 .

IPCC (Intergovernmental Panel on Climate Change), (1990) Impacts assessment of climate change-report of Working Group II. Canberra, *Australian Government Publishing Service*.

WB (World Bank),(2000). Bangladesh, Climate Change and Sustainable Development. Report No 21104-BD

REMOVAL OF CR(VI) FROM AQUEOUS SOLUTION USING BONE CHAR AS A LOW COST ADSORBENT

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ABSTRACT

Chromium contamination is one of the most noteworthy environmental problems over the last few decades. The worst chromium poisoning has been noted in areas where electroplating, tannery, automobile, pigments, paper and pulp, fertilizer, textile, steel, and metal finishing industries are located. Chromium concentration in wastewater should not exceed 0.05 mg/L for Cr(VI) according to the International environmental standards. Toxicity and carcinogenicity are the two significant phenomena that chromium has as a heavy metal. The health effects of hexavalent chromium [Cr(VI)] include stomachs dysfunction, ulcers, respiratory problems, immune systems weakness, kidney and liver damage, lung cancer, anemia, alteration of genetic material, muscle breakdown, blood clotting abnormalities, eye damage, etc. Consequently, the objective of the present study was to evaluate the effectiveness of bone char as an adsorbent for Cr(VI) removal from wastewater. Bone char is a granular solid material, black in color, and prepared by calcining cattle bones at a specific temperature (450 ^{0}C). The particle size of bone char used was in between 53 and 250 µm. Batch sorption experiments were conducted to optimize the experimental parameters such as pH, contact time, initial Cr(VI) concentration, and bone char dosages. The initial Cr(VI) concentrations were varied between 10 mg/L and 1000 mg/L during batch tests. The concentrations of Cr(VI) were measured by using a graphite furnace equipped atomic absorption spectrophotometer (GF-AAS) with an air-acetylene flame and chromium hollow cathode lamp at a slit of 2 nm and wavelength of 357.9 nm. The results demonstrate that about 100 % of Cr(VI) was removed at pH 1 with initial Cr(VI) concentration of 10 mg/L using 2 g of bone char dosage in 50 mL of solution volume after 3 hours of sorption reaction. The maximum adsorption capacities of bone char was 6.46 mg/g for initial Cr(VI) concentration of 1000 mg/L after 5 hours of reaction time..

Keywords: Contamination; Hexavalent chromium; Bone char; Batch test; Sorption.

1. INTRODUCTION

Chromium is considered as an abundant transition metal in the world [1]. Over the last few decades, chromium contamination is one of the most noteworthy environmental problems because of its increasing concentration as a consequence of different human activities. Industrial application of chromium are the major human activities which involves electroplating, tannery, automobile, pigments, textile, steel, and metal finishing industries. Chromium containing industrial effluents contaminates nearby atmosphere, soil, and water.

Hexavalent chromium is the major oxidation states that has been detected in industrial effluents. In groundwater around chromium ore processing facilities, Cr(VI) concentration of up to 91 mg/L has been reported [2]. Concentration of Cr(VI) in the effluent of electroplating industry has been found to range from 3 to 50 mg/L [3]. The concentration of chromium in some municipal and industrial wastes was reported to be as high as 1993 mg/kg [2]. However, Cr(III) and Cr(VI) concentration should not exceed 5 and 0.05 mg/L, respectively in wastewater [4]. Moreover, standard limit of chromium for humans is 0.05 µg/L for drinking water, and 0.5 – 500 µg/m^3 for atmospheric exposure [5].

The health problems associated with Cr(VI) include dermatological, respiratory problems, stomachs dysfunction, ulcers, kidney and liver damage, alteration of genetic material, lung cancer, asthmatic bronchitis, bronchospasms, etc. [5, 6, 7].

The common treatment methods of hexavalent chromium removal include *precipitation* [8, 9, 10], *ion exchange* [11, 12, 13, 14, 15], *membrane separation* [16, 17, 18, 19], *reduction and electrochemical reduction* [9, 20]. On the other hand, most of the above mentioned treatment technologies have limitations which include: generation of toxic sludge, high operational and maintenance expenditure, high energy requirements, too long treatment time, and poor removal efficiency [1, 20].

But, adsorption process has been implemented significantly for the removal of metal ions from wastewater which process several benefits include simple, effective, and economical contaminant removal with less amount of sludge production [21, 22, 23]. Among the various existing adsorbent, activated carbon has been widely implementing for the past few decades [21, 22, 23]. In order to adsorb Cr(VI) from wastewater, reported low cost adsorbents include *maple saw dust* [24], *bentonite and expanded perlite* [25], *chitin and chitosan* [26], *coal* [27], and important bio-sorbents include *Nymphaea rubra* [28], *Pseudomonas aeruginosa* and *Bacillus subtilis* [29], *Mucor racemosus* [30], *Pseudomonas spp* [31], *Bacillus spp* [32]. However, as a low cost sorbent, bone char has rarely been investigated [33, 34, 35]. In the study, bone char has been used to evaluate pH, Cr(VI) concentration, and bone char dosage.

2. METHODOLOGY

2.1 Bone char

Bone char was purchased from the Anthracite Filter Media (AFM), Culver City, California, USA. It is a granular solid material, black in color, and prepared by calcining cattle bones at a specific temperature (450 ^{0}C). Particle size of purchased bone char used was in between 53 and 250 µm.

2.2 Sorption procedure

In this study, a batch sorption process was employed to find out the optimum pH, bone char dosage, and initial Cr(VI) concentration. The stock solution of 1000 mg/L of Cr(VI) was prepared by dissolving 2.8269 g of potassium dichromate ($K_2Cr_2O_7$) salt (Sigma-Aldrich LTD, USA, product code no 101064403) in deionized water. The stock solution of Cr(VI) was subsequently diluted using deionized water to the required working concentration prior to use in the batch tests. The pH variation was conducted by either 15.8M HNO_3 or 1M NaOH. The Cr(VI) and bone char mixture was placed on the shaker at 150 rpm at room temperature.

About 4 mL samples were collected using separate syringe at 0, 0.5, 1, 2, 3, 4, 5, 6, 7, 8, and 24 hours and 2 mL/day from day 2 to day 6. Collected samples were then filtered through 0.45 µm syringe filter. Filtered samples were stored in glass bottle and preserved in the refrigerator at 4°C with sample pH below 2 to avoid the degradation of Cr(VI) concentration.

Samples were then analyzed to measure the Cr(VI) concentrations using a graphite furnace equipped atomic absorption spectrophotometer (GF-AAS) with an air-acetylene flame and chromium hollow cathode lamp at a slit of 2 nm and wavelength of 357.9 nm. Prior to analyze the sample, the GF-AAS instrument was calibrated regularly.

2.2.1 Evaluation of optimum parameters (pH, bone char dosage, and initial Cr(VI) concentration) and equilibrium studies

The optimum pH was determined by changing solution pH of 1, 2, 3, 5, 7, and 9 with bone char dosage of 2 g/50 mL solution. Moreover, different bone char dosages of 0.5, 1, 2, and 3 g in 50 mL solution were evaluated to know the optimum dosage at pH of 1. The initial Cr(VI) concentration was kept at 10 mg/L to investigate optimum pH and bone char dosage. The initial Cr(VI) concentrations of 5, 10, 20, 50, 70, and 100 mg/L were evaluated with 2 g of bone char in 50 mL solution at pH 1 in order to obtain its optimum value.

The Cr(VI) concentrations at equilibrium (C_e) and at any particular time (C_t) were evaluated for different initial Cr(VI) concentrations (C_0) of 5, 10, 20, 50, 100, 300, 400, and 1000 mg/L maintaining the pH and bone char dosage constant at the optimum value. These concentrations (C_0, C_e, C_t) were applied to calculate the sorption capacity of bone char at a particular time (q_t) and at equilibrium (q_e) according to the mass balance Eq. (1) and Eq. (2), respectively [1]. Afterward, these sorption capacities ($q_{e,}$ q_t) were used to draw the equilibrium sorption isotherms and kinetics model plots.

$$q_t = \left(\frac{C_0 - C_t}{m}\right)V \qquad (1)$$

$$q_e = \left(\frac{C_0 - C_e}{m}\right)V \qquad (2)$$

where C_0 is the initial Cr(VI) concentration in mg/L, C_t is the Cr(VI) concentration at a particular time in solution in mg/L, C_e is the equilibrium Cr(VI) concentration in mg/L, V is the volume of the solution in litter (L), and m is the dry mass of bone char in gram (g).

3. ILLUSTRATIONS

3.1 Evaluation of the optimum sorption parameters

3.1.1 Effects of pH

The Cr(VI) removal efficiencies decreased consistently with increased pH from 1 to 9 and were very slightly increased with time for each pH (Fig. 1). The Cr(VI) removal efficiency was found to be around 85 % after 30 min of sorption reaction and was increased to 98 % after 120 min at pH of 1. The removal efficiencies were around 60 %, 45 %, 43%, 35 %, and 32 % at pH of 2, 3, 5, 7, and 9, respectively. Since the highest removal efficiency was obtained at the lowest pH, the optimum pH was considered as 1. Another study has also recommended the same optimum pH [33].

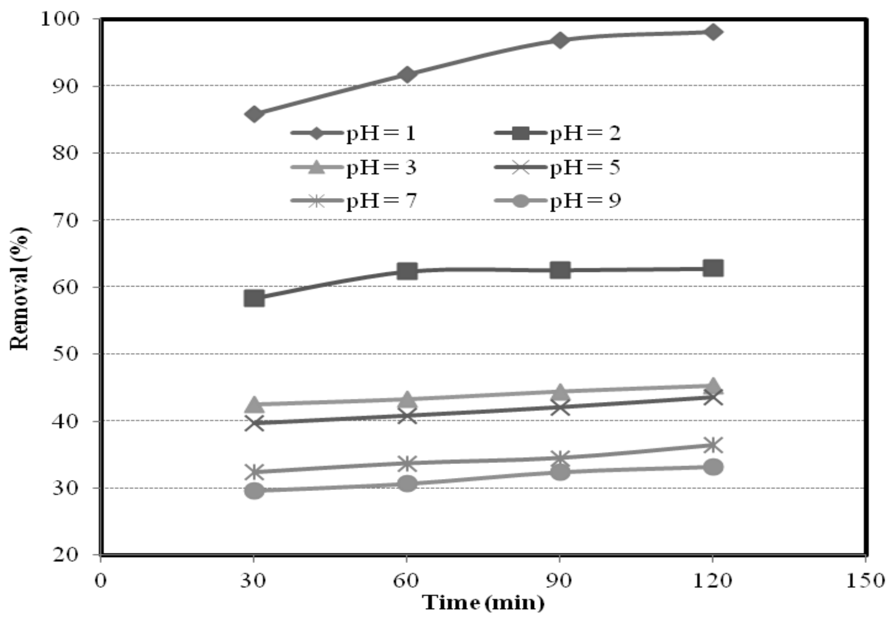

Figure 1: Removal efficiencies of Cr(VI) at various times with different pH values using bone char as a sorbent. Solution volume: 50 mL; initial Cr(VI) concentration: 10 mg/L; amount of bone char: 2 g.

3.1.2 Effects of bone char dosages

The increase in bone char dosage from 0.5 g to 3 g leads to sharp increases in removal efficiencies as expected (Fig. 2). The highest percentage of Cr(VI) removal was achieved for the highest studied bone char dosage of 3 g. At this dosage, 95 % Cr(VI) removal was observed after 30 min, and the removal improved to 100 % after 90 min. The removal efficiencies were in the range of 57% to 66 % for a bone char dosage of 1 g, and from 43 % to 60 % for a bone char dosage of 0.5 g. Since the removal efficiencies were similar (100 %) for bone char dosages of 2 g and 3 g after 120 min, 2 g of bone char per 50 mL of solution was considered as the optimum dosage. Another study has also recommended the same optimum dosage value [33].

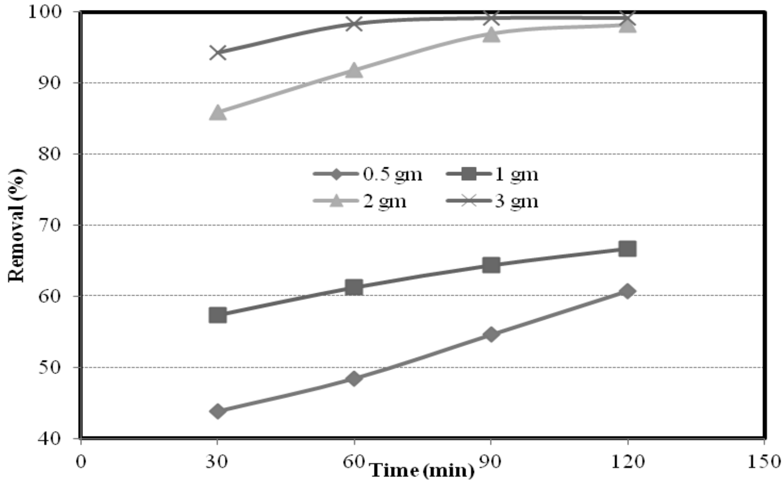

Figure 2: Removal efficiencies of Cr(VI) at various times with different dosages of bone char as a sorbent. pH: 1; solution volume: 50 mL; initial Cr(VI) concentration:10 mg/L.

3.1.3 Effects of initial Cr(VI) concentration

The highest Cr(VI) removal efficiency (100 %) and the lowest Cr(VI) removal efficiency (72 %) were achieved for the highest studied initial Cr(VI) concentration of 100 mg/L and the lowest studied initial Cr(VI) concentration of 5 mg/L after 120 min of sorption reaction (Fig. 3). In case of other initial Cr(VI) concentrations between 5 and 100 mg/L, the Cr(VI) removal efficiencies were deteriorated continuously with rising initial Cr(VI) concentrations. The removal efficiencies were almost 100 % for both the initial Cr(VI) concentration of 5 and 10 mg/L after 120 min of sorption reaction. Therefore, 10 mg/L of Cr(VI) was considered as the optimum initial concentration. Another study has also recommended the same optimum value [33].

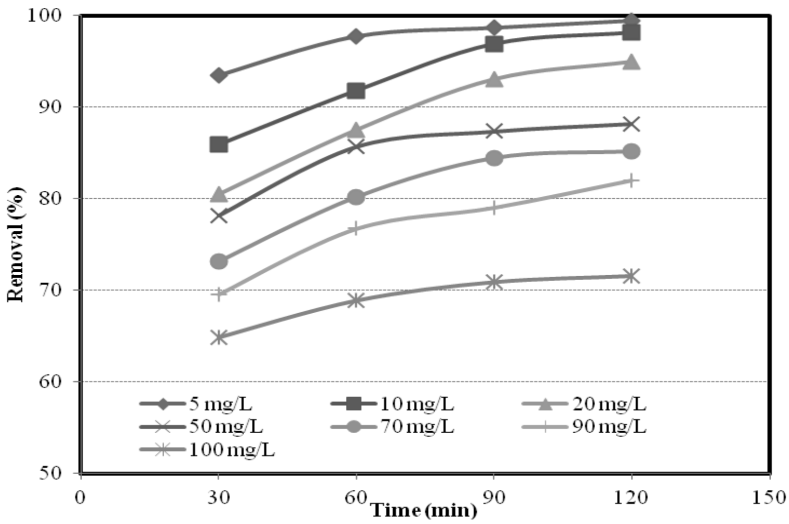

Figure 3: Removal efficiencies of Cr(VI) at various times with different initial Cr(VI) concentration using bone char as an adsorbent. pH:1; solution volume: 50 mL; amount of bone char: 2 g.

3.2 Evaluations of the sorption capacities

The sorption capacities (q_t) improved with an increase in the initial Cr(VI) concentrations (5-1000 mg/L) until reached at equilibrium values (q_e) and the time required to achieve the equilibrium sorption capacity was also increased except 1000 mg/L. The lowest sorption capacity of 0.148 mg/g was found after 3 hours for the lowest studied initial Cr(VI) concentration of 5 mg/L but the highest sorption capacity of 6.46 mg/g was obtained after 5 hours for the highest studied initial Cr(VI) concentration of 1000 mg/L. The sorption capacities reached a constant level at equilibrium value of 0.264, 0.466, 1.389, 1.905, and 2.508 mg/g for initial Cr(VI) concentrations of 10, 20, 50, 70, and 100 mg/L, respectively from 8 hours to 6 days. For initial Cr(VI) concentration of 300 mg/L and 400 mg/L, the equilibrium sorption capacity was observed later as 5.801 and 6.057 mg/g, respectively after 24 hours.

4. CONCLUSIONS

In this study, 100 % Cr(VI) was removed at optimum value of pH, bone char dosage, and initial Cr(VI) concentration of 1, 40 g/L, and 10 mg/L, respectively. The highest sorption capacity was 6.46 mg/g for initial Cr(VI) concentration of 1000 mg/L.
In future work bone char could be treated in the acidic medium prior to use in the sorption process. The real industrial wastewater sample could be used rather than synthetic wastewater which does not contain other metal ions, microorganism, and humic acid.

REFERENCES

[1] Mohan, D. & Pittman, C. U. (2006). “Activated carbons and low cost adsorbents for remediation of tri- and hexavalent chromium from water”. Journal of Hazardous Materials B137: 762–811.
[2] Kabata-Pendias, A. and Mukherjee, A. B. (2007). “Trace Elements from Soil to Human”
Springer-Verlag Berlin Heidelberg, New York. ISBN-10 3-540-32713-4 and ISBN-13 978-3-540-32713-4.
[3] Daniel, R., Anjaneyulu, Y., and Krupadam, R. J. (2009). “Cr(VI) removal from electroplating industrial effluents: A greener and cheaper method”. Zastita Materijala, 50:13-18.
[4] Pugh, C. T., Palmer, T. T., and Carr, P. A. (2008) “A study of significant variables and their effects on the analysis of hexavalent chromium”. J. Chem. Health Saf., 15 (3), 25.
[5] Barceloux, D. G. & Barceloux, D., (1999). Chromium. Clinical Toxicology, 37(2): 173–194.
[6] Al-Jabari, M., Abualfailat, M., Shaheen, S. (2012). ‘Treating leather tanning wastewater with stone cutting solid waste”. Clean-soil, air, water, 40 (2): 206-210.
[7] Pellerin, C. and Booker, S.M., (2000). “Reflections on hexavalent chromium: health hazards of an industrial heavyweight”. Environ Health Prospect. 2000 September; 108(9): A402–A407.
[8] Kurniawan, T. A., Chan, G. Y. S., Lo, W. H., and Ba-bel, S. (2006). “Physico-chemical treatment techniques for wastewater laden with heavy metals, Chemical Engineering Journal, 118, 83–98.
[9] Roundhill, D. M., Koch, H. F. (2002). “Methods and techniques for the selective extraction and recovery of oxoanions”. Chem. Soc. Rev. 31:60–67
[10] Mohan, D., Rajput, S., Singh, V. K., Steele, P. H., Pittman, C. U. (2011). “Modeling and evaluation of chromium remediation from water using low cost bio-char, a green adsorbent”. Journal of Hazardous Materials,188: 319–333.
[11] Tiravanti, G., Petruzzelli, D., Passino, R. (1997). “Pretreatment of tannery wastewaters by an ion exchange process for Cr(III) removal and recovery”. Water Sci. Technol. 36: 197–207.
[12] Rengaraj, S., Yeon, K. H., Moon, S. H. (2001). “Removal of chromium from water and wastewater by ion exchange resins”. J. Hazard. Mater. 87 (1–3): 273–287.
[13] Rengaraj, S., Joo, C. K., Kim, Y., Yi, J. (2003). “Kinetics of removal of chromium from water and electronic process wastewater by ion exchange resins: 1200H, 1500H and IRN97H”. J. Hazard. Mater. 102 (2/3): 257–275.
[14] Petruzzelli, D., Passino, R., Tiravanti, G. (1995). “Ion exchange process for chromium removal and recovery from tannery wastes”. Ind. Eng. Chem. Res. 34: 2612–2617.
[15] Rivero, M.J., Primo, O., Ortiz, M.I. (2004). Modelling of Cr (VI) removal from polluted groundwaters by ion exchange”. J. Chem. Technol. Biotechnol, 79:822-829.
[16] Kozlowski, C. A., Walkowiak, W. (2002). “Removal of chromium(VI) from aqueous solutions by polymer inclusion membranes”. Water Res. 36: 4870–4876.

[17] Shaalan, H., Sorour, M., Tewfik, S. (2001). "Simulation and optimization of a membrane system for chromium recovery from tanning wastes". Desalination 14: 315–324.
[18] Barakata, M. A. and Schmidt, E. (2010). "Polymer-enhanced ultrafiltration process for heavy metals removal from industrial wastewater". Desalination, 256: 90-93.
[19] Ozaki, H., Sharmab, K. and Saktaywirf, W. (2002). "Performance of an ultra-low-pressure reverse osmosis membrane (ULPROM) for separating heavy metal: effects of interference parameters". Desalination, 144: 287-294.
[20] Martinez, S.A. & Rodriguez, M.G. (2007) "Dynamical modeling of the electrochemical process to remove Cr (VI) from wastewaters in a tubular reactor". J. Chem. Technol. Biotechnol, 82: 582-587
[21] Zhonghua, H., Lin, L., Yijiu, L., Yaming, N. (2003). "Chromium Adsorption on High-Performance Activated Carbons from Aqueous Solution". Sep. Purif. Technol. 31(1),13.
[22] Mohan, D., Singh, K. P., Singh, V. K. (2005). "Removal of hexavalent chromium from aqueous solution using low-cost activated carbons derived from agricultural waste materials and activated carbon fabric cloth". Ind. Eng. Chem. Res. 44: 1027–1042.
[23] Mohan, D., Singh, K. P., Singh, V. K. (2006), "Trivalent chromium removal from wastewater using low cost activated carbon derived from agricultural waste material and activated carbon fabric cloth" J. Hazard. Mater. 135: 280–295.
[24] Yu, L. J., Shukla, S. S., Dorris, K. L., Shukla, A., Margrave, J. L. (2003) "Adsorption of Chromium from Aqueous Solution by Maple Saw Dust". J. Hazard. Mater. 100 (1–3): 53.
[25] Chakir, A., Bessiere, J., Kacemi, K. E., Marouf, B. (2003). "A Comparative Study of the Removal of Trivalent Chromium from Aqueous Solutions by Bentonite and Expanded Perlite". J. Hazard. Mater. 95 (1–2): 29.
[26] Mcafee, B. J., Gould, W. D., Nadeau, J. C., Costa, A. C. A. (2001). "Bio Sorption of Metal Ions Using Chitosan, Chitin, and Biomass of Rhizopus oryzae". Sep. Sci. Technol. 36 (14): 1520.
[27] Lakatos, J., Brown, S. D., Snape, C. E. (2002). "Coals as sorbents for the removal and reduction of hexavalent chromium from aqueous waste streams". Fuel 81: 691–698.
[28] Taj, R. Y., Renganathan, S., Partha, N., Baskar, G., Velan, M., (2009). Removal of chromium from synthetic effluent using Nymphaea rubra. Clean-Soil Air Water, 37: 787.
[29] Tarangini, K., Kumar, A., Satpathy, G. R., Sangal, V. K., (2009). Statistical optimization of process parameters for Cr(VI) biosorption onto mixed cultures of Pseudomonas aeruginosa and Bacillus subtilis. Clean-Soil Air Water, 37: 319.
[30] Jabasingh, S. A. & Pavithra, G., (2010). Response surface approach for the biosorption of Cr(vi) ions by mucor racemosus. Clean-Soil Air Water, 38: 492.
[31] Mondaca, M. A., Gonzalez, C. L., Zaror, C. A., (1998). Isolation, characterization and expression of a plasmid encoding chromate resistance in Pseudomonas putida KT2441.Lett. Applied Microbiology, 26: 367.
[32] Campos, J., Maartinez-pacheco, M., Cervantes, C., (1995). Hexavalent chromium reduction by a chromate resistant Bacillus sp strain. Antonie van Leeuwenhoek, 68: 203.
[33] Dahbi, S, Azzi, M., Guardia, de la M. (1999). "Removal of Hexavalent Chromium from Wastewaters by Bone Charcoal". Fresenius J. Anal. Chem. 363: 404.
[34] Chojnacka, K. (2005). "Equilibrium and kinetic modelling of chromium(III) sorption by animal bones". Chemosphere 59 (2005) 315–320.
[35] Wilson, J. A., Demis, J., Pulford, I. D. and Fendorf, S. (2001). "Sorption of Cr(III) and Cr(VI) by natural bone charcoal". Environmental Geochemistry and Health, 23: 291–295.
[36] Langmuir, I. (1916). "The constitution and fundamental properties of solids and liquids". Part I. Solids. J. Am. Chem. Soc. 38, 2221.
[37] Freundlich, H.M.F. (1906). "Uber die Adsorption in Losungen". Z. Phys. Chem. 57(A), 385.
[38] Lagergren, S., 1898. About the theory of so-called adsorption of soluble substances. Kungliga Svenska Vetenskapsa-kademiens. Handlingar, 24(4):1-39
[39] Ho, Y. S., McKay, G. (1998). "Pseudo-second order model for sorption processes". Process Biochemistry 34 (1999) 451.
[40] Dong, X., Ma, L. Q., Li, Y. (2011). "Characteristics and mechanisms of hexavalent chromium removal by bio char from sugar beet tailing". Journal of Hazardous Materials190, (2011): 909-915.

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STUDY ON DECENTRALIZED WASTEWATER TREATMENT PLANT IN PAANCHTALA COLONY KHULNA

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ABSTRACT

In Bangladesh the centralized waste collection and management run by the City Corporation Authority of the Government cannot meet up to the needs of required collection and management of waste generated throughout the country. This problem can be solved by introducing decentralized wastewater treatment plants (DEWAT). The NGO Nabolok funded by UK Aid has taken an initiative by installing two decentralized wastewater treatment plant at Paanchtala colony, Khalishpur, Khulna. The installed decentralized wastewater treatment plant operates without mechanical means and consists of a settler, anaerobic baffled septic tank, filter bed of gravel, sand, plantation-beds and a pond. The pH of the outlet water has been found to be 6.89 from the tests which is acceptable according to Bangladesh Water Quality Standards for Surface Water. The other parameters such as TSS is 1680 for inlet and 105 for outlet, TDS is 3920 for inlet and 1180 for outlet, COD is 6320 for inlet and 1090 for outlet, BOD5 is 580 for inlet and 39 for outlet, DO is 0.89 for inlet and 0.72 for outlet and the temperature decreased from 30 degree to 22.3 degree. This study verifies the success of the program and recommends application of it for other communal places.

Keywords: Title, abstract, objective, results, conclusions, recommendations

1. INTRODUCTION

Soils are primary media for construction work and it's bearing capacity is generally governed by its strength and In Bangladesh the quality and coverage of wastewater collection and service provision is generally not at all enriched and systems available for wastewater management are mostly ineffective. Most wastewater and sludge collection and treatment is city based and centralized. These centralized systems are run by government organizations. These available systems struggle to keep pace with the rate of urban development. Hence many communities suffer from environmental health related problems. The decentralized wastewater treatment plant can be a very good solution for this. UK Aid funded NGO Nabolok has constructed decentralized waste water treatment plant for the local citizens of the colony to give them a better environment friendly living. This study has been conducted to collect details of all the construction related work, cost, and success of the program of the decentralized waste water treatment plant of Peoples Paanchtala Colony, Khulna. In this project existing design and construction procedure of the decentralized wastewater treatment plant was studied and several parameters were tested to find out whether the program is a success or not. There were no sewerage systems or dumping place where people could through their solid waste. So they used to dump their wastage beside their residence. It was being accumulated day by day. All the toilets were unhygienic and permanently damaged due to lack of proper maintenance. Lack of hygiene and toilet facilities also lead to an open air toilet tendency. As a result the place was full of wastage and polluting the environment. All septic tanks and drainage system are also same condition. As a result human excreta are malted with mud, water and foot path. After testing water quality, it has been found that wastewater contained BOD limit 250-750, where normal limitation is not over 50. The NGO Nabolok repaired the expired hygiene and toilet facilities and constructed the decentralized wastewater treatment plant. The area is at a distance from the city. So, the decentralized wastewater treatment plant is a pretty good solution.

2. OBJECTIVES OF THE STUDY

a) To conduct field survey in the decentralized wastewater treatment plants (DEWAT) at Paanchtala Colony, Khalishpur, Khulna.
b) To study the field performance of the DEWAT.

c) To collect the influent and effluent wastewater samples from DEWAT and perform laboratory analysis for water quality.
d) To study possibility and scope of DEWAT for its implementation in other communities.

3. METHODOLOGY

Steps to be followed in this project work:
Firstly, a preliminary field investigation was done in the decentralized wastewater treatment plant (DEWAT) at Paanchtala colony, Khulna. Two specific buildings were selected for the sample collection recognized by D3 & D4 building. Influent and effluent wastewater samples were collected at different stages of treatment operations in the decentralized wastewater treatment plant (DEWAT) for both D3 & D4 buildings.. The tests of pH, BOD_5, COD, TSS, TDS, Faecol Coliform were performed in laboratory. After conducting the tests, the results were analyzed, compared and presented in graphs for both D3 & D4 building.

3.1 Field Investigation

The unhygienic scenarios of the other adjacent buildings are shown in Figure 1. This was the past scenario of the D3 and D4 buildings which are taken under the decentralized wastewater treatment system. Figure 2 shows the constructions of decentralized waste water treatment plant of D4 building.

Figure 1: Scenario of Adjacent Buildings

Figure 2: Construction of Decentralized Wastewater Treatment Plant of D4 Building

3.2 Sample Collection

Samples from six points of decentralized wastewater treatment plant were collected. These six points are outlet of planted filter, inlet of settler, outlet of settler, middle of anaerobic filter bed, outlet of anaerobic baffled reactor and outlet of polishing pond. The samples were collected in sterilized plastic bottles and immediately after collecting samples they were brought in environmental engineering laboratory. In Figure 3 some photos of sample collection sample collection area are shown below. In these figures the plan of the wastewater treatment plant, planted filter and polishing pond is shown.

Figure 3.3: Data Collection Area

3.3 Laboratory Test

Samples were collected from the six points of the wastewater treatment plant. Important parameters like pH, TDS,SS, faecal Coliform, DO, COD, BOD_5were tested for different selected points of the plant. Figure 4 shows the laboratory tests.

Figure 4: Test of Collected Samples

3.4 Flow chart

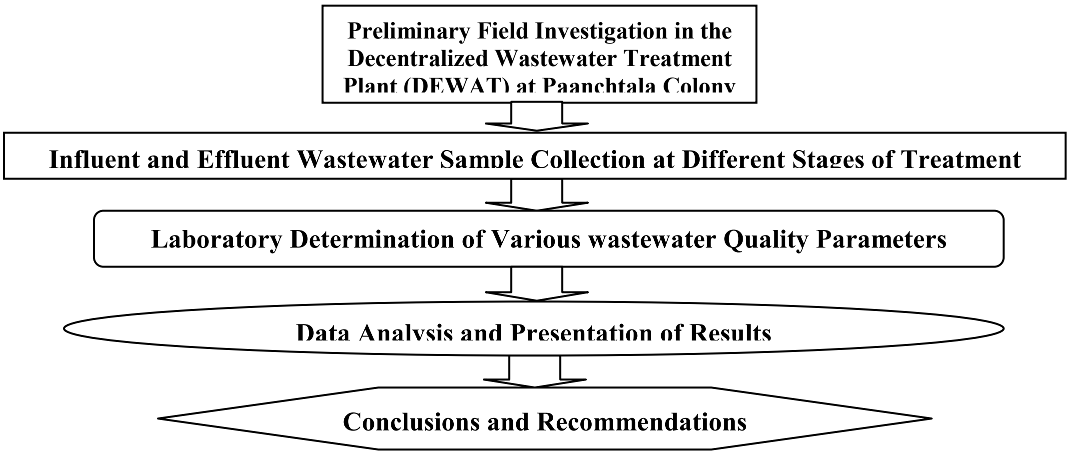

Figure 5: Flow chart of the methodology

4. RESULTS AND DISCUSSION

Table 9: Data of laboratory tests of D3 building

Sl no.	Parameters	Units	Limits for Disposal in Water Bodies (Bangladesh Gadget,1997)	Inlet of Settler	Outlet of Settler	Middle of Anaerobic Filter Bed	Outlet of Anaerobic Baffled Reactor	Outlet of Planted Filter	Outlet of Polishing Pond
1	**pH**	--------		7.01	6.81	6.93	6.98	7.01	6.89
2	**BOD_5**	mg/l	40	580	163	158	133	74	39
3	**COD**	mg/l		6320	4290	3310	2360	2090	1090
4	**TSS**	mg/l	100	1680	180	170	130	105	105
5	**TDS**	mg/l		3920	1920	1610	1490	1390	1180
6	**Faecal Coliform**	N/100ml	1000	1050	650	630	470	420	300
7	**DO**	mg/l		0.89	0.72	0.69	0.74	0.69	0.72
8	**Temperature**	^oC	30	20.2	22	22.1	22.1	22.1	22.3

Variation of Wastewater Quality Parameters of Different Locations of Treatment Plant for D3 Building

Table 1 shows the variation of pH of six selected points of DEWATS. The pH is found to be the least in the outlet of settler. The value of pH is finally decreased to standard limit. It shows the variation of BOD5 of six selected points of DEWATS. The BOD5is found to be in the descending order. The COD is found to be in the descending order. The TSS is found to be in the descending order. The Faecal Coliform is found to be in the descending order

Table 2: Data of laboratory tests of D3 building

Sl no.	Parameters	Units	Limits for Disposal in Water Bodies (Bangladesh	Inlet of Settler	Outlet of Settler	Middle of Anaerobic Filter Bed	Outlet of Anaerobic Baffled Reactor	Outlet of Planted Filter	Outlet of Polishing Pond
1	**pH**	--------	--------	6.8	6.5	6.45	6.5	6.6	6.8
2	**BOD5**	mg/l	40	232	180	120	102	80	29
3	**COD**	mg/l	-------	4100	3000	2300	2200	1400	1400
4	**TSS**	mg/l	100	1400	50	60	50	60	60
5	**TDS**	mg/l	------	1290	1100	1050	1020	960	890
6	**FC**	N/100ml	1000	800	400	450	470	310	260
7	**DO**	mg/l	------	0.60	0.52	0.47	.0.69	0.72	1.1
8	**Temperature**	oC	30	23.5	23.6	23.7	23.6	23.7	23.9

Performance of Decentralized Wastewater Treatment Plant in Different Time Interval

Data of treated water was provided by the project manager for four consecutive months. They are plotted in following figures to show variation in different time interval.

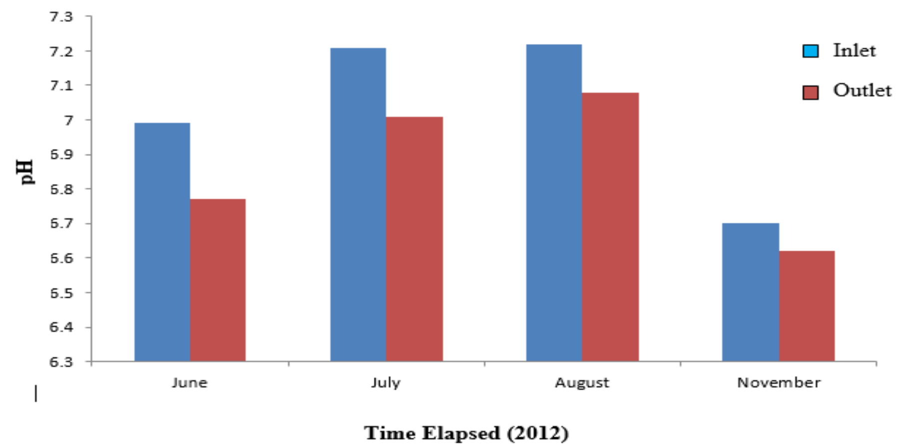

Figure 6: Variation of pH with time

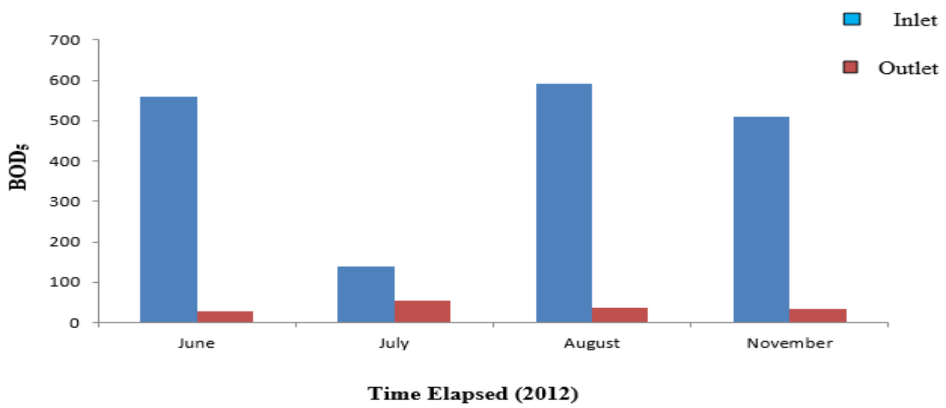

Figure 7: Variation of BOD5 with time

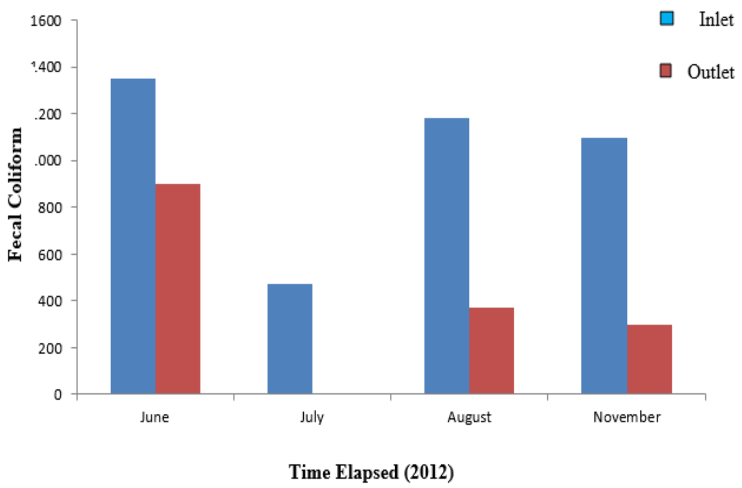

Figure 8: Variation of Faecal Coliform with Time

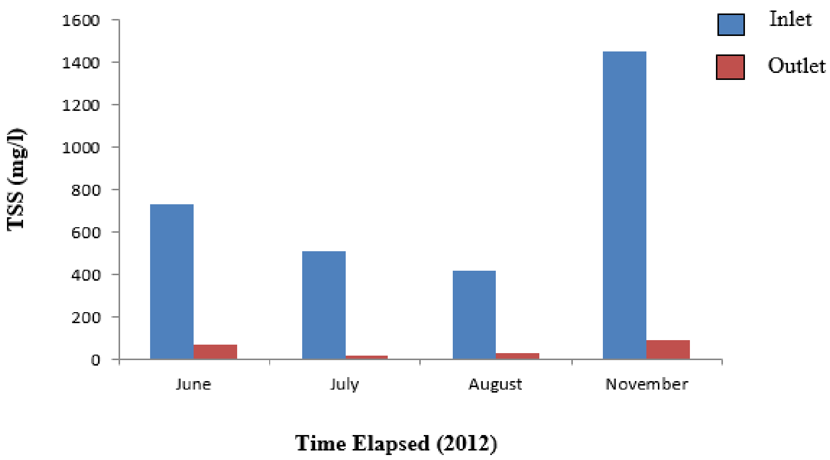

Figure 9: Variation of TSS with Time

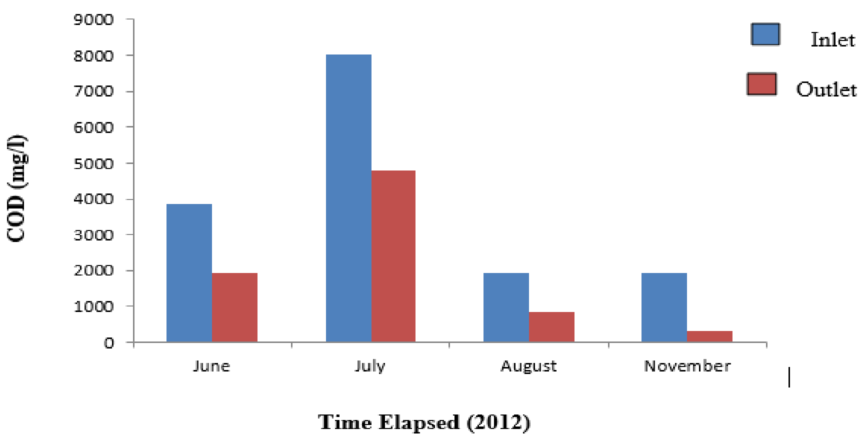

Figure 10: Variation of COD with Time

5. CONCLUSIONS

Concerning the first objective, it was observed that the overall waste management system is very poor. Though City Corporation Authority is trying to handle the whole task of waste collection from bazaar but due their resource limitation it becomes delay to collect the wastes properly. Regarding to the second objective, from questionnaire survey and field visit the entire project was studied. Eighty four families dwell in the D building and around twenty four lakh bdt was required for the project. According to the third objective, necessary laboratory experimental setup was installed. Samples were collected and tested. To fulfill the last objectives, it is proposed that the decentralized wastewater treatment plant is a feasible and helpful solution for communities and it can be also well applied in KUET. A properly designed decentralized wastewater treatment plant would be a sustainable solution with respect to environment and economy.

6. RECOMMENDATIONS

Within the scope of this study, we aimed to address exclusively the effect of improper disposal of sludge and now it becomes essential to find a sustainable solution of this improper disposal. Treating the wastewater in a decentralized process enables the community to have a hygienic and environment friendly life. This also lessens the load to the central system available. This is a cost effective project if done on a large scale.

REFERENCES

APHA, AWWA, WPCF (1998). Standard method for the examination of water andwastewater.*20th Edition.* Washington DC.

Asano, T (1998). Wastewater reclamation and reuse. Water quality management library.10.Pennsylvania. USA. Technomic publishing company.

Bangladesh Bureau of Statistics, (1997) Statistical Year Book of Bangladesh (19th ed.), Bureau of Statistics, Ministry of Planning, Gov. of peoples Republic of Bangladesh.

Crites and Tchobanoglous (1998). Small and Decentralized Wastewater ManagementSystems. McGraw-Hill.

Documentations of NGO Nabolok, funded by UK Aid.

Deshmukh, S., Gupta, R., Agrawal, V.S., 2002. Improving the solid wastemanagement by developing the peoples perception – a case study.International Waste Management Biennial Congress & Exhibition, Durban.

EPA Decentralized Wastewater Treatment -a Sensible Solution, MOU Intro Paper.

KnowledgeAreas/DecentralizedSystems/LatestNews/Smart_Clean_and_Gree.aspx.

U.S. Environmental Protection Agency Office of Wastewater Management Decentralized Program – www.epa.gov/owm/onsite

Water Environment Research Foundation Decentralized Systems.

Web Address: – http://www.werf.org/i/a/k/DecentralizedSystems.aspx

Water Aid (1997) Water Aid Annual Review 1997, Water Aid, London.

Water Environment Research Foundation's Smart, Clean and Green: 21st Century Sustainable Water Infrastructure – http://www.werf.org/i/c/.

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DEVELOPMENT OF AN ARSENIC AND IRON REMOVAL UNIT USING SAND FILTER WITH CHARCOAL AND IRON POWDER

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ABSTRACT

Contamination of arsenic is a major concern for our country. In Bangladesh groundwater is the primary source of drinking water. But unfortunately 60% of water from tubewells has arsenic above 10 ppb (WHO guideline). Nowadays, arsenic has become an acute problem around the world. In the context of prevalence of high concentrations of arsenic in tubewell water, a wide range technologies have been tried for the removal of arsenic from groundwater. The most common technologies utilized the conventional processes of oxidation, co-precipitation and adsorption onto coagulated flocs, adsorption onto sorptive media, ion exchange and membrane techniques for arsenic removal. Some technologies utilized indigenous materials for arsenic removal. The conventional technologies have been scaled down to meet the requirements of households and communities and suit the rural environment. The rural people habituated in drinking tubewell water may find arsenic removal from tubewell water as a suitable option for water supply. In many arsenic affected areas, arsenic removal may be the only option in the absence of an alternative safe source of water supply. In Bangladesh, iron is also found at high level. People are reluctant to drink iron contaminated water mainly due to bad taste. Available iron removal techniques are aeration, coagulation, gravel bed flocculation, sedimentation, filtration, etc. But sometimes water is contaminated with arsenic and iron both. As Bangladesh a developing country there is a need to develop a cost effective and easy process to remove arsenic and iron using locally available materials for using in small system or home treatment units. Iron powder and charcoal has been found to be very effective in arsenic and dissolved iron removal from contaminated groundwater. Iron oxyhydroxide surface absorbs arsenic species i.e. arsenite and arsenate by forming complexes with the surface sites. Arsenic strongly attracts to sorption sites on the surfaces of iron solids and eventually strains out effectively through Arsenic-Iron Removal Filter (AIRF). In this study, the AIRF filter was designed and its efficiency and life time were investigated. The filter was found successful to remove both the arsenic and iron from highly contaminated groundwater.

Keywords: Adsorption and co-precipitation, arsenic-iron removal filter, groundwater, treatment efficiency.

1. INTRODUCTION

Groundwater is available in shallow aquifers in adequate quantity in the flood plains for development tubewell based water supply for scattered rural population. Bangladesh achieved remarkable successes by providing drinking water at low-cost to the rural population through sinking of shallow tubewells in flood plain aquifers. Unfortunately arsenic contamination of shallow tubewell water in excess of acceptable limit has become a major public health problem. Thousands of people have already shown the symptoms of arsenic poisoning and several millions are at risk of arsenic contamination from drinking tube well water (Ahmed, 2002). Hence, provision of arsenic free water is urgently needed to mitigate arsenic toxicity and protection of health. On the other hand Groundwater collected through handpump tubewells in Bangladesh generally shows a high concentration of iron, and in most locations the concentration is much higher than the acceptable limit 1 mg/L (Ahmed and Rahman, 2003). Although, iron does not cause any direct health problem, due to aesthetic reasons, rural people generally refuse to use tube well water in iron problem areas and they become more inclined to use the unprotected surface water sources. Here a small filter is developed to remove iron and arsenic from groundwater using iron powder, charcoal and Sylhet sand. Iron powder and charcoal has been found to be very effective in arsenic and dissolved iron removal from contaminated groundwater. Iron oxhydroxide surface absorbs arsenic species i.e. arsenite and arsenate by forming complexes with the surface sites. Arsenic strongly attracts to sorption sites on the surfaces of iron solids and eventually strains out effectively through Arsenic-Iron Removal Filter (AIRF).

2. METHODOLOGY

2.1 Research Strategy

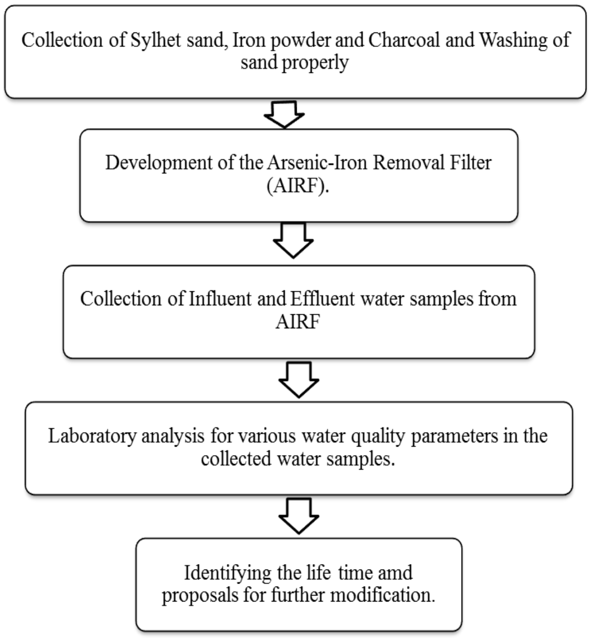

Figure 1: Research Strategy of AIRF

2.2 Design of AIRF

Here a Arsenic-Iron Removal Filter was designed and constructed. It was constructed of 2 buckets. The layer used in 1st bucket from top is 4 inch of sand, 2 inch of charcoal, 2 inch of iron powder and 4 inch of sand. In the 2nd bucket only 8 inch of sand layer was used. The schematic diagram of the AIRF unit is shown in figure below:

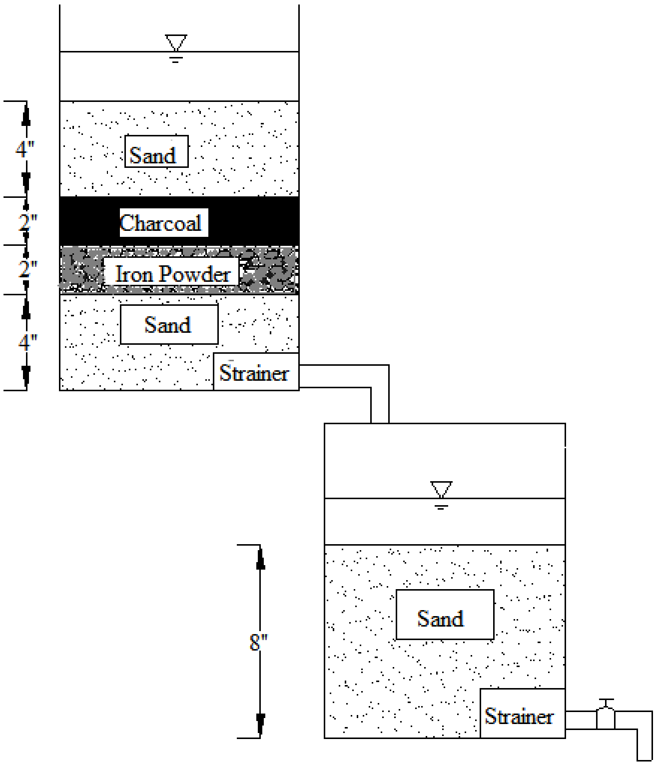

Figure 2: Line Diagram of AIRF

2.3 Unit Process of AIRF

A brief description of the methodology that was followed in conducting the study is given below

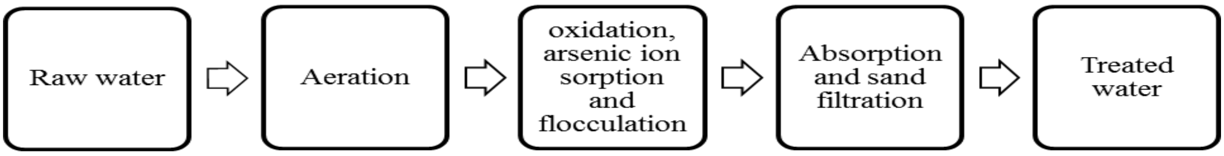

Figure 3: Flow Diagram of the Unit Processes of the AIRF

The AIRF is consists of two buckets and these two buckets are used in such a way that they are connected through a PVC pipe and one bucket lies below the another while water pass through from upper to lower bucket.

2.3.1 1st Bucket (down-flow Charcoal, Iron powder and Sand flocculation):

Oxidation and subsequent precipitation of iron oxyhydroxides on the top and within the interstices of sand media absorbs arsenic oxyanions. Iron oxyhydroxide surface absorbs arsenic species i.e. arsenic and arsenate by forming complexes with the surface sites. Arsenic is strongly attracted to sorption sites on the surfaces of iron solids. Sinusoidal flow through activated charcoal enhances collisions for the flocculation of precipitated particles and eventually strains out effectively toward the 2nd bucket.

2.3.2 2nd bucket (down-flow Sand Filter):

Final removal of precipitated particles both through sorption onto iron oxyhydroxides and mechanical straining take place during down-flow through the sand filtration process in the 2nd bucket.

3. RESULTS AND DISCUSSION

Detail laboratory analysis and tests were carried out through AIRF to investigate the change in some important water quality parameters. Although various parameters are being analyzed but the removal efficiency of arsenic and iron is the major concern. Highly arsenic and iron contaminated water were filtered through the filter.

4. ARSENIC REMOVAL EFFICIENCY

Before filtration the arsenic contamination of the raw water was ranges from 350-500 ppb. From the Figure 4.1, it is seen that the filter is very efficient for the removal of arsenic. At the beginning of filtration the removal efficiency of arsenic is 100%. But after a while its efficiency is decreased. From the Figure 4.1, it is clear that the filter can treat water up to 250 liter. After that the filter needs backwashing for further removal of arsenic.

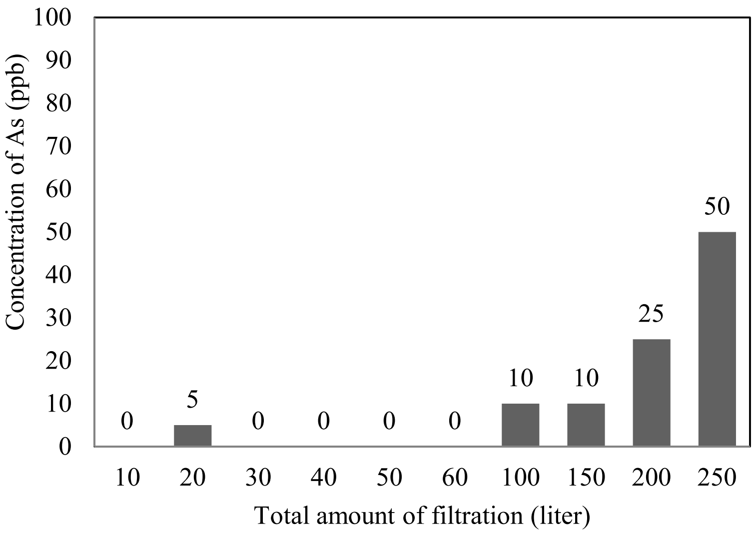

Figure 3: Removal Efficiency of Arsenic

4.1 Iron Removal Efficiency

Appreciable iron removal efficiency was notified from the filter. At the initial stage, the concentration of iron was ranges from 5-6mg/L. From the Figure 4.2, it is seen that after the filtration the concentration of Iron were less than 0.67mg/L which was within the allowable limit. The removal efficiency of Iron is increasing day by day.

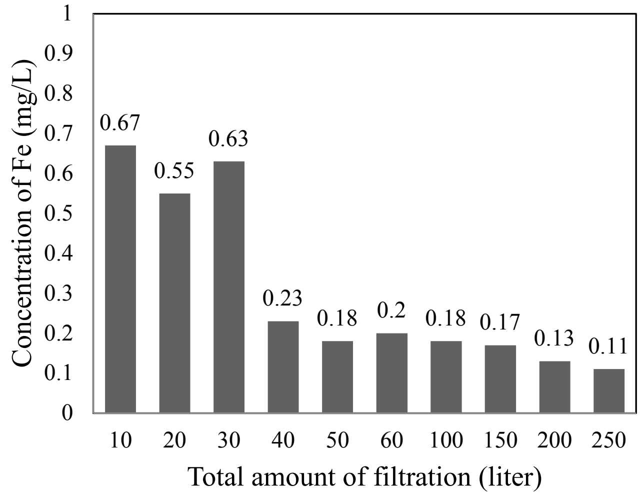

Figure 4: Iron Removal Efficiency

4.2 Influences over Other Parameters

Although the main focus for constructing AIRF is to remove arsenic and iron but besides this influence over other important water quality parameters are also measured.

4.2.1 Chloride

Chloride is found in variable amount in groundwater. Chloride may present naturally in groundwater and may also originate from diverse sources such as weathering, leaching of sedimentary rocks and infiltration of seawater etc. The maximum permissible limit of chloride in potable water is 150-600 mg/L. The amount of chloride of raw water is 600-700 mg/L. But the filter can not remove the salinity of water. From the Figure 4.3, it is seen that there are a variety of concentration of chloride in filtered water this is because of the variation of concentration of chloride in raw water.

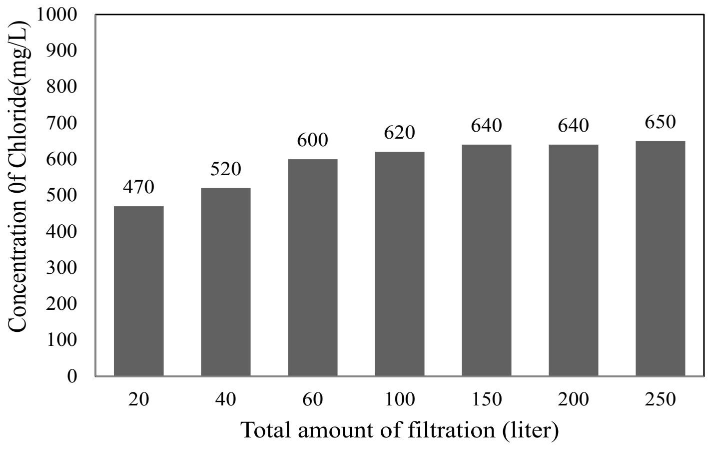

Figure 5: Removal Efficiency of Chloride

4.2.2 Total Coliform

Total coliforms are a group of bacteria can be found both in faeces and the environment. Total coli form of raw water was around 2 nos./100 mL. From the Figure 4.4, it is seen that after filtration the amount of TC were in between 1-2 nos./100mL. Drinking standard for TC is 0 mg/L. So the results found that after filtering the samples do not satisfy the Bangladesh standard.

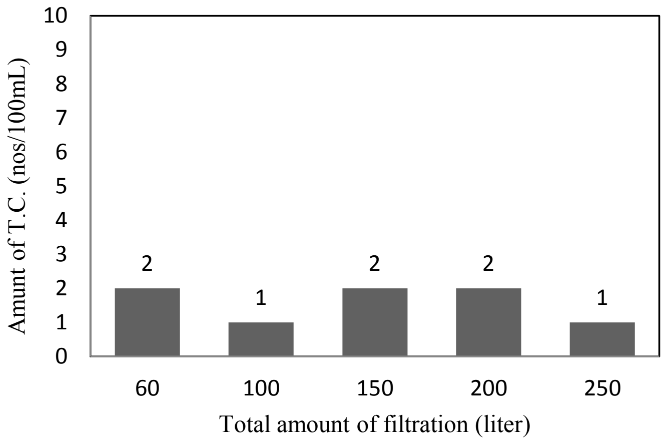

Figure 6: Removal Efficiency of Total Coliform

4.2.3 E. Coli

E. coli is a species of feacal coliform bacteria that is specific to feacal material from humans and other warm-blooded animals. Figure 4.5. shows the filtered water satisfies the Bangladesh standard which is 0 mg/L.

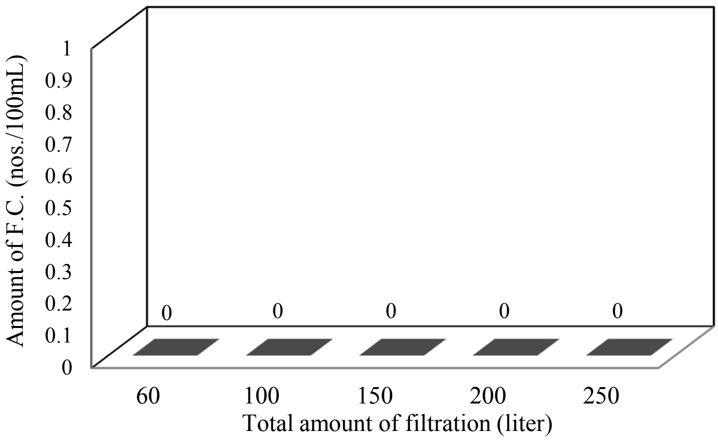

Figure 7: Removal efficiency of E.Coli

5. CONCLUSIONS

This study is conducted with the development of a small household filter with locally available materials. Detailed laboratory model analysis and tests were carried out to determine some important design parameters and the backwashing period of the AIRF for practical application. In this study, about 300 liter of arsenic contaminated water was passed through the filter and found that the removal efficiency of arsenic was decreased day by day. So, after some period washing of the filter material is needed. The major findings of the study are outlined as below:

- This filter can remove nearly 100% arsenic and 95% iron from the contaminated water by using locally available material without adding chemicals.
- This is a rapid filtering process. So it does not need much time to treat water.
- The filter can treat 250 liter of highly contaminated water. After that it needs proper washing of the filter materials.

REFERENCES

Ahmed, M.F. (2001). "An overview of Arsenic Removal Technologies in Bangladesh and India." In: Bangladesh University of Engineering and Technology, (Editor): Technologies for Arsenic Removal from Drinking Water. Dhaka, pp. 251-269.

Ahmed, M.F., Rahman, M.M., (2003). "Water Supply and Sanitation, Rural and Low Income Urban Communities", 2nd edition, Dhaka, Bangladesh.

IMPACT OF WATER LOGGING ON AGRICULTURE AND FOOD SECURITY: A CASE STUDY IN SATKHIRA, BANGLADESH

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ABSTRACT

An integrated drainage path is very important for tropical countries. When the drainage path can not wok properly, water logging must be arised in the surroundings. For the same three southwestern coastal districts of Bangladesh namely Satkhira, Khulna and Jessore experiencing severe prolonged water logging and remain submerged for long periods every year, especially during the monsoon season. Which have a significant affect on agriculture production systems and overall economic and social development of the area . As a result of that effect many people have been bound to change their occupation and leave their homestade. Consequiently the yearly population growth rate become 0.115% in the study area where as the overall national growth rate of the country is 1.078%, Which is ten times less than the country's growth rate.This situation calls for an emergency action for the sustainable development of the country.

Keywords: Water logging, agriculture, monsoon, rainfall, coastal areas

1. INTRODUCTION

Bangladesh is one of the most populated agro based developing countries of the world. It is situated in the active monsoon region of the world. It is deltaic land with total area of about 1,47,570 sq. km and the population is 14, 97, 72,364(BBS, 2012). About 80% of the total population is involved in agriculture as a profession. Rice, jute, tea, sugarcane etc are main agriculture products in Bangladesh. Rice is the main food of the country. Due to the increasing pressure of population together with limitations of natural resources, it has become very urgent to ensure the maximum production of food. Different types of natural hazards such as flood, drought, cyclone and storm surge, tidal surges, intrusion of saline water, increase of soil salinity and river water salinity, water-logging, tidal flooding, river bank erosion, tornadoes etc., significantly affect the agriculture production systems and overall economic and social development of the country (BCAS, 2010). Coastal areas are the most susceptible region of this country. Majority of the people in coastal areas are involved in crop cultivation and fishing and they remain frequently unemployed due to tidal flooding and other natural disasters resulting food insecurity in the areas (Hossain, 2010). Three south-western coastal districts of Bangladesh namely Satkhira, Khulna and Jessore are the worst hit areas and experiencing severe prolonged water logging and remain submerged for long periods every year, especially during the monsoon season (Oxfam, 2011 & UNDP, 2011).

This study aims at investigating the impact on agriculture and also mitigating measure of prolonged water-logging in the south-western coastal zone of Bangladesh in particular the worst affected Tala upazila under Satkhira district hence proposing strategies for its sustainable development.

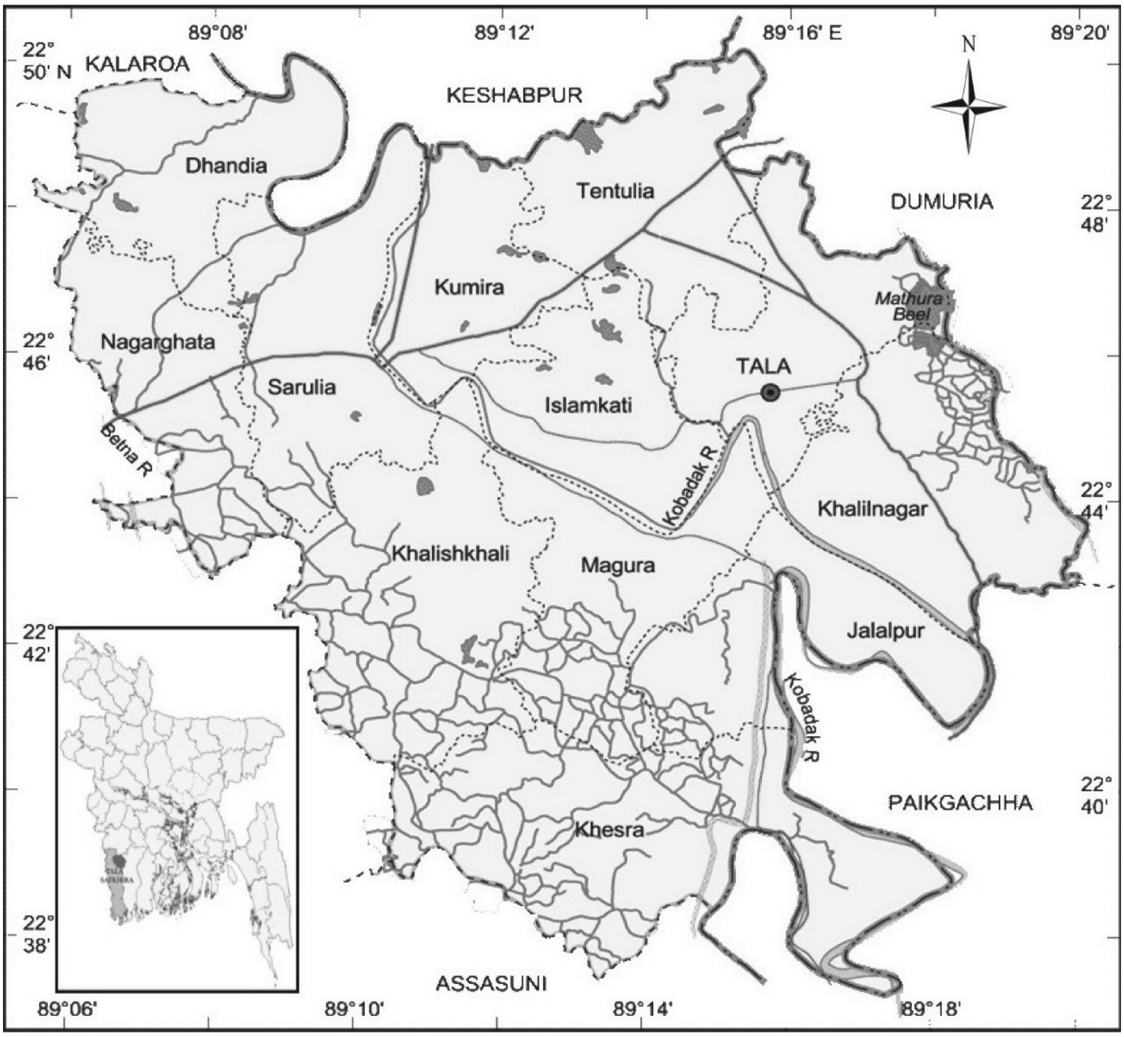

Figure 1: Location Map of the Study Area

The upazila occupies an area of 337.24 sq. km. It is located between 22°32' and 22°50' north latitudes and between 89°05' and 89°20' east longitudes. The upazila bounded on the north by Keshabpur upazila of Jessore zila and Kalaroa upazila, on the east by Dumuria and Paikgachha upazilias of Khulna zila, on the sourth by Assasuni upazila and on the west by Satkhira Sadar upazila (BBS, 2012).

2. DATA USED

Both primary and secondary data collected in conducting this research. A field survey and open discussion conducted with the involvement of local people of the severe water logged areas of the study area. The agricultural data, as base line information and progress with compare to base line, collected from respective Tala Upazila Agriculture Office. Other necessary data and information collected from different government organizations. In addition, a range of journals, reports and previous study reports thoroughly analyzed.

2.1 Rainfall Scenario of Adjacent of the Study Area

Bangladesh is a tropical country and located on the extensive floodplains of the Ganges and Brahmaputra. The Himalayas stands to the northeast of the country and the Bay of Bengal lies on the south of the country. As a result heavy downpour occurs on the country, especially in the coastal areas of our country.

Table 1: Year wise Total Rainfall (mm) in Monsoon over Khulna Division

Years	**Rainfall (mm)**				
	June	**July**	**August**	**September**	**Monsoon**
2000	1279	1989	1621	1499	6388
2001	1776	1248	731	1015	4770
2002	3162	1701	1694	1374	7931
2003	1541	1281	1113	1161	5096
2004	1890	1475	2148	2565	8078
2005	1415	1891	961	1744	6011
2006	1439	2219	1573	1922	7153
2007	1876	2629	1065	1534	7104
2008	1299	1828	1193	1593	5913
2009	847	1456	2213	1696	6212
2010	1355	1000	872	1000	4227

Source: Bangladesh Meteorological Department, 2011

2.2 Water Level Scenario of Adjacent River of the Study Area

The study area is situated in the Ganges Basin, Kabodak river is the main drainage path of this upazila which is an offshoot branches out from the Bhairab river and flowing south meets with the Shibsa river near Paikgachha in Khulna district. A prolong flooding situation was prevailed along the Kobodak river in September and October in 2012 (figure 2). At Jhikorgacha, the water level flowed above the danger level for continuous 49 days with a peak of 4.39mPWD on 20th September, which was 28cm above the danger level (4.11m). As a result, part of Satkhira, Khulna and Jessore districts were flooded for prolong period (BWDB, 2012). In 2011 the water level flowed above the danger (BWDB, 2011) for continuous 89 days with a peak of 4.99m on 21st August, which was 88cm above the danger level (4.11m) at the same point. As a result, part of Satkhira, Khulna and Jessore districts were flooded for prolong period. At Jhikorgacha, the water level of river Kobodak crossed the danger level on 8th August and remained above the danger limit till 5th of November. This is due to the poor drainage condition and excessive rainfall in the region.

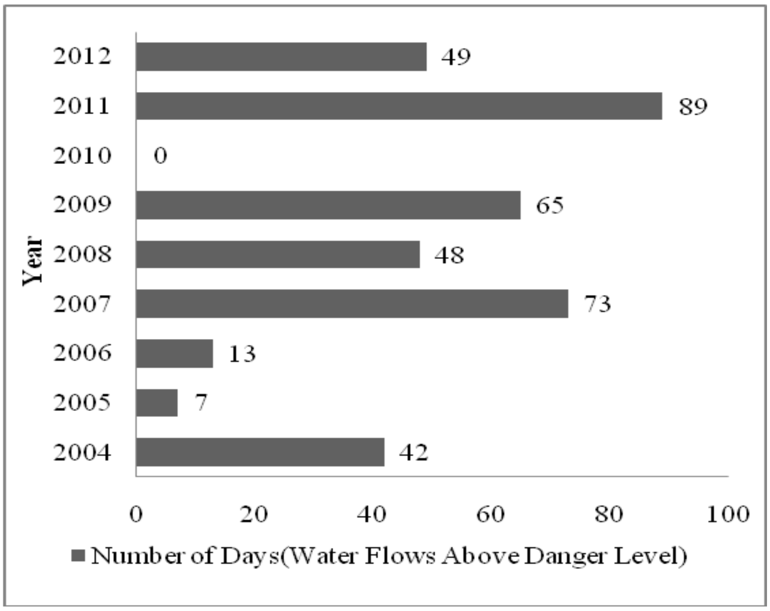

Figure 2: Water Flow above Danger Level (DL) in the Kabodak River

From figure2, it is clearly visible that, except 2010, the Kobodak flowed above its danger level at Jhikorgacha in every year since 2004. For this reasons the surrounding area of Kabodak river remain water logged for five to six months every year.

2.2.1 Causes of water logging in the study area

Climate change often identified as the reason for increased floods and water logging situation in Bangladesh. However, the reality is more complex and caused by the interaction of various factors. Main factors that have contributed to the current dire situation listed below:

i. Incessant monsoonal rains
ii. Lack of adequate drainage path
iii. Lack of maintenance of embankments built along the rivers in the 1960s
iv. Increased sediment load and siltation of rivers
v. Restricted river flow due to embankments built for shrimp farming along the coast
vi. Release of water in monsoon from barrages in India (notably Farakkah Barrage and Durgapur/Damodar Barrage)

2.3 Rice Production Scenario of The Study Area

Three different kinds of rice are cultivated in three different times in the study area round the year. They are Aus, Aman and Boro. The Aus rice planted in the month of March-May and harvested in the month of June-August. Aman rice planted in the month of May-August and harvested in the month of November-January. Similarly, Boro rice planted in the month of December-February and harvested in the month of April-May.

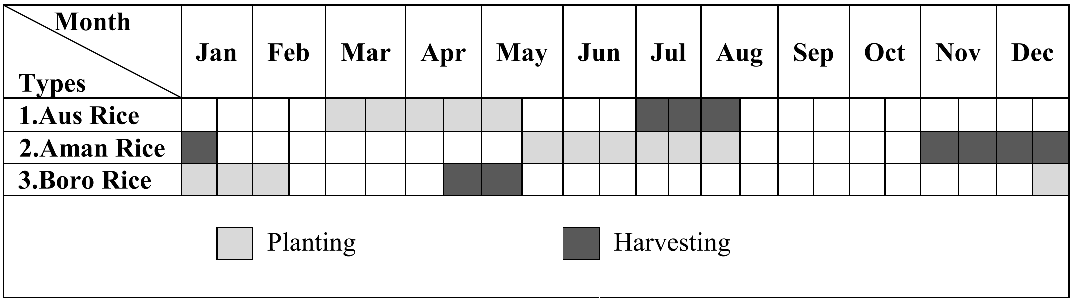

Figure 3: Cropping Calendar of the Study Area

Aus and Aman rice production is mostly dependent on rainwater of the monsoon but the Boro rich is wholly predominant by irrigation in the winter season. For this reason Aus and Aman rice production is seriously fluctuating with the climatic condition.

Table 2: Year-wise Area under Cultivation in the Study Area

Sl. No.	Name of Rice	Area under Cultivation(ha)				
		Year				
		2007-2008	2008-2009	2009-2010	2010-2011	2011-2012
1	HYV Aman	10250	15790	7092.85	14880	8700
	Local Aman	1550	300	1600	1270	700
	Total Aman	**11800**	**16090**	**8693**	**16150**	**9400**
2	HYV Aus	348	2360	1131	1795	1500
	Local Aus	2	40	5	10	10
	Total Aus	**350**	**2400**	**1136**	**1805**	**1510**
3	HYV Boro	11320	11550	1280	1295	15250
	Hybrid Boro	4300	4700	15375	15520	1250
	Local Boro	0	0	0	0	0
	Total Boro	**15620**	**16250**	**16655**	**16815**	**16500**
Total Rice		**27770**	**34740**	**26484**	**34770**	**27410**

Source: Upazila Agriculture Office, Tala, 2013.

Table 3: Year-wise Cleaned Rice Production in the Study Area

Sl. No.	Name of Rice	Cleaned Rice Production(MT)				
		Year				
		2007-2008	**2008-2009**	**2009-2010**	**2010-2011**	**2011-2012**
1	HYV Aman	25625	40004	19859.98	42408	24969
	Local Aman	1736	435	1760	1778	1050
	Total Aman	**27361**	**40439**	**21620**	**44186**	**26019**
2	HYV Aus	755.16	5664	2589.99	4020.8	3450
	Local Aus	2.3	60	6.95	14.6	19.5
	Total Aus	**757.46**	**5724**	**2597**	**4035.4**	**3470**
3	HYV Boro	43582	43890	5760	5827	60237.5
	Hybrid Boro	23177	21150	59962.5	60528	5000
	Local Boro	0	0	0	0	0
	Total Boro	**66759**	**65040**	**65722.5**	**66355**	**65237.5**
Total Rice		**94877.46**	**111202.97**	**89939**	**114576.4**	**94726**

Source: Upazila Agriculture Office, Tala, 2013.

For easy and clear detection of rice production situation of the study area, a Yearly Rice Production graph showed below:

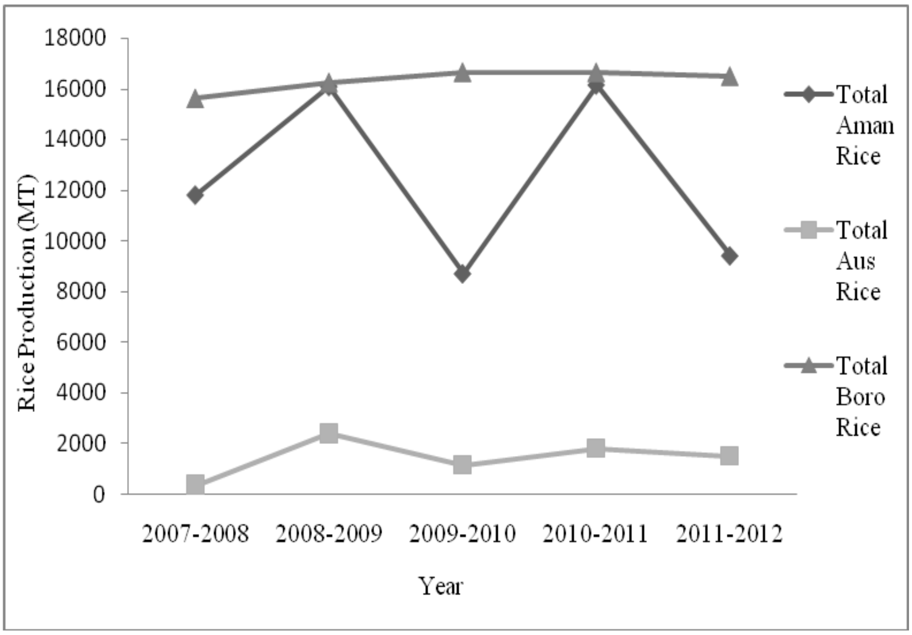

Figure 4: Yearly Rice Productions in the Study Area

From Table 3 and Figure 4 it is easy to say that the Aman rice production became half in each alternative year in the study area.

3. RESULTS AND DISSCUSSION

As a result, of prolonged water logging the economic condition of the study area is going to befall poorer to poorer. On the other hand, the local people did not have any work that they can live on. Many of them forced to change their occupation or leave their origin homestead. This picture is clearly visible in BBS census report 2011. Based on BBS; in the study area there were 307695 people in 2001 and after ten years in 2011 its total population became 311236 (BBS, 2012). The yearly population growth rate of the study area is 0.115%, where as the overall national growth rate is 1.078%. If this situation is stays for long time, the study area population will decreased significantly.

Table 3: Population of the Study Area

Sl. No.	Area	Population		Changes	Yearly Growth Rate (%)
		2001	2011	2001-2011	
1	Study Area	307695	311236	3541	0.115
2	Bangladesh	130030000	144043697	14013697	1.078

4. CONCLUSIONS

Based on the above discussion it is understandable that water level rising in the Kabodak river is the main cause for prolonged water logging. Water logging affects severely by reducing agricultural production, which consequently deteriorates socio-economic condition, since many people forced to leave their homestead forever from the study area.

REFERENCES

BBS, (2012) Population and Housing Census 2011, Community Report, Satkhira Zila, June 2012, Bangladesh Bureau of Statistics, Statistics and Informatics Division, Ministry of Planning, Bangladesh.

BCAS, (2010) Assessing Long-term Impacts of Vulnerabilities on Crop Production Due to Climate Change in the Coastal Areas of Bangladesh, Prepared as part of the National Food Policy Capacity Strengthening Programme, November, 2010 by Bangladesh Center for Advanced Studies (BCAS), Dhaka, p.6-13.

Oxfam, (2011) Monsoon Floods and Water Logging in Bangladesh, August 2011, Situation Report 01, p.1.

UNDP, (2011) Water Logging in Satkhira District: An Analysis of Gaps between Needs and Response, November 2011, Early Recovery Facility, UNDP Bangladesh, p.4.

Hossain, M. A.(2010) Indigenous Technology for Adapting to Water Logging Situation for Sustainable Livelihood Security in Low Lying Areas of Bangladesh, Soil Resource Development Institute, Ministry of Agriculture, Krishi Khamar Sarak, Farmgate, Dhaka-1215, Bangladesh.

BWDB, (2012) Annual Flood Report 2011, Flood Forecasting and Warning Centre, Bangladesh Water Development Board,WAPDA Building, Motijheel C/A, Dhaka-1000.

BWDB, (2011) Annual Flood Report 2012, Flood Forecasting and Warning Centre, Bangladesh Water Development Board,WAPDA Building, Motijheel C/A, Dhaka-1000.

Upazila Agriculture Office, Tala (2013) Base Line Information and Progress with Compare to Base Line.

CHARACTERIZATION, FLOW DYNAMICS AND TREATMENT OF WASTE WATER OF TEXTILE DYEING VAT AT KUSHTIA DISTRICT IN BANGLADESH

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ABSTRACT

Textile Dyeing Vat, a part and parcel of Textile Dyeing Industries has been disposing waste water at Kumarkhali Upazilla in Kushtia District for more than four decades. Directly or indirectly, these Dyeing Vats situated in Kumarkhali Suburban areas have been deteriorating the Environmental Condition through polluting the surface water by direct mixing and ground water by infiltrating of high toxicity waste water. The specific objective of this study was to characterize of the waste water of the textile Dyeing Vat in order to realize the overall condition of the industrial area. Besides this objective, Flow rate measurement to know the quantity of wastewater generation and treatment to optimize the chemical dose were another objectives of the study. Turbidity, Color, pH, TDS, TSS, Cr^{6+}, BOD_5,SO_4^{2-} and COD were the Basis for Characterization. In terms of Turbidity, Color and pH, Spectrophotometer, Turbidity Meter and pH meter were used. Besides for other parameters, Standard methods were used to measure the values. Bucket method was selected for the flow rate measurement. Jar test method for coagulation process was used to determine the optimum dose in terms of removal of pH, Color, Turbidity, and rest of all parameters while Ferric Chloride was used as the Chemical Coagulant. The pH, Color, Turbidity,BOD_5,COD,TDS, TSS, SO_4^{2-}, and Cr^{6+} was 8.14, 5170 Pt-Co,452 NTU, 233mg/L, 381mg/L, 2601 mg/L, 408.60 mg/L,751.35 mg/L and 0.2 mg/L in the month of June and 8, 5050Pt-Co, 449.29NTU, 213.948 mg/L,372 mg/L, 2540mg/L, 400mg/L,750 mg/L and 0.2 mg/L in the month of July respectively during the year of 2012. The wastewater flow rate was 874.5 Lt/hr at the peak hour of the Textile Dyeing Vat. For Treatment. Ferric Chloride itself achieved the removal efficiencies for pH by 24.16%, Turbidity by 98.86%, Color by 99.14%, TDS by 33.85%, TSS by 100%, COD by 85.07%, SO_4^{2-} by 6.75%, Cr^{6+} by 85% and BOD_5 by 89.59%. In Conclusion, the study provided the pathways by which Environmental Management System for the Textile Dyeing Vat areas will get basic clues and information through getting nature of Vat waste water, optimization of Ferric Chloride dose for treatment and quantification of waste water flow.

Keywords: Vat Dye, waste water, TDS, flow rate, turbidity.

1. INTRODUCTION

The nature of the Characterization, Flow Dynamics and Treatment of the wastewater of the Textile Dyeing Vat indicates the pollution level and removal efficiencies respectively directly or indirectly of all the textile industries situated at Kumarkhali Upazilla in Kushtia District in Bangladesh. Besides, the scope of the research is increase of awareness to the owners of the industries as well as people of the municipality about environmental pollution, designing common effluent treatment plant etc. Kumarkhali is an important centre of the Bangladesh Handloom Board; about one hundred automated textile manufacturing units are located here. Main exports of kumarkhali Upazilla is Textile products. Textile Dyeing Vat is an unit of all textile Industries that are used to have the cotton colored in different Vat dyes. Being a part and parcel of the large industries of kumarkhali Upazilla among all industries situated in kumarkhali Upazilla,Textile Dyeing Vats discharge waste water. Textile industries in Bangladesh are denoted as the red colored industries because of serious environmental pollution. It is reported that textile effluent is very low in terms of LC50 and exhibits very high toxicity with acute toxicity unit (ATU) levels between 22 and 960. Dye baths surfactants and fibers could have high levels of BOD, COD, color, toxicity, surfactants, fibers, turbidity and contain heavy metals. The result of a textile waste water treatment process aims at the destruction of the waste water color by means of coagulation/flocculation techniques using Ferrous Sulfate or lime. Due to the use of various chemicals and auxiliaries in the processing of textiles, large volume of waste water with numerous pollutants is discharged every day. They have strong rules regarding these types of industries including the preset up of Effluent Treatment Plant (ETP) with the given time period to get the Environmental Clearance Certificate which is mandatory to get utility connections like gas, water etc. The first step to investigate the problem of the waste water discharged by the industry was done by using a Dissolved Oxygen Meter, pH meter, Thermometer, Dissolved oxygen bottle etc especially based on standard methods. On the other hand, as a first footprint on this specific research in such a study area in Bangladesh, the treatment phase has been done by using Ferric Chloride as coagulant for chemical treatment. Actually the Dissolved oxygen of the waste water has been found 0.3 mg/L and 0.4 mg/L as well. To treat the waste water, the Ferric Chloride has been used as 400 mg/L approximately. The principle result of waste water characterization shows that except pH, all the parameters like color, turbidity, Total Dissolved solid, Total Suspended Solid, chromium as a heavy metal, sulfate, Biological Oxygen Demand, Chemical Oxygen Demand have been exceeded the standard limit of Department of Environment, Government of Bangladesh. The Waste Water Flow rate was 874.5 lt/hr at the peak hour of the Textile Dyeing Vat. For Treatment, as the waste water is less concentrated comparing to the newly developed textile industrial waste water in Dhaka city, it takes less amount of chemical for treatment.

2. METHODOLOGY

The overall methodology consists consequently like topic selection, conceptualization, study area selection, primary data collection, secondary data collection, data processing and preparation of a conference paper. In Characterization phase, Turbidity meter, Color meter and pH meter are used for measuring turbidity and color and pH respectively. Besides for measuring Total Dissolved Solid, Total Suspended Solid, Biological Oxygen Demand, Chemical Oxygen Demand, Standard methods (APHA, AWWA, and WEF, 1995) was used. Sampling design was based on purposive sampling in this research work. Traditional Bucket Method was used for measurement of the Flow Dynamics. In treatment phase, Jar test method for Ferric Chloride coagulation was used to treat the textile waste water. On the other hand, computer software like Microsoft office was used for preparation the graph and data processing.

3. RESULTS AND DISCUSSION

3.1 Wastewater flow Dynamics of the Textile Dyeing Vat

Wastewater flow from the textile dyeing vat begins at 10am till 6.30 pm at Kumarkhali Upazilla in Kushtia District. It is variable in such kind of wastewater generation source like Textile Dyeing Vat. Because of the

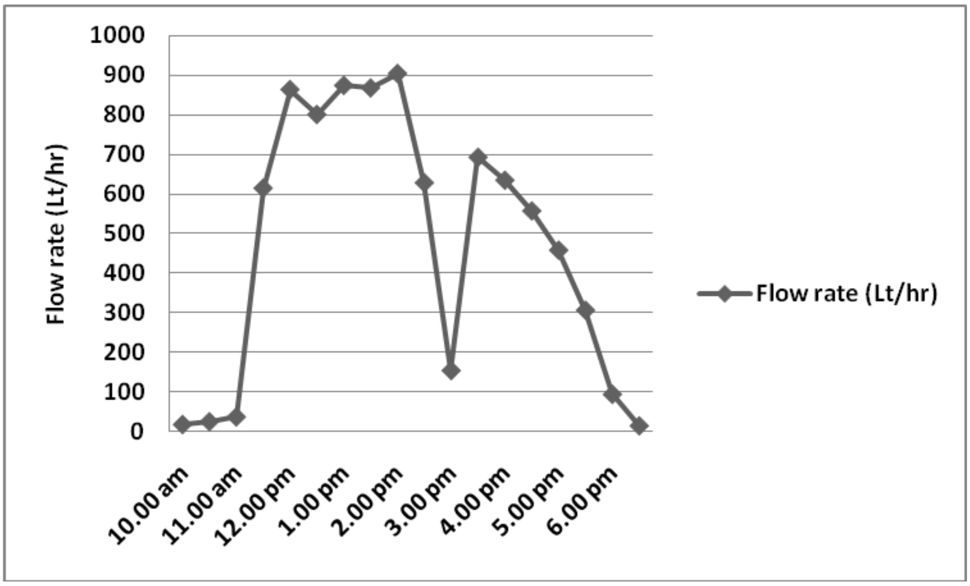

Figure 15: Flow rate of the Wastewater of the Textile Dyeing Vat

arrival of the textile dyeing Vat labours in Vat area from different corner of the upazilla Kumarkhali as well as the supplied cotton that are coloured by dyes act as the variables for the beginning of the Textile Dyeing Vat. Actually the textile wastewater flow dynamics incorporates the variation of flow rate during the continuation of the Dyeing Vat. The flow rate was initially low because of the less supply of cotton in the industry at the time of 10am. Besides, labours could not reach at the beginning time except someone. The wastewater of the dyeing Vat is discharged through a mud canal. Mud canal ends at the small drain that is connected with the sub drain of the municipality of Kumarkhali Upazila in Kushtia District. During the flow rate measurement of textile vat wastewater, it has been used stopwatach, buckets of different volumes like 15, 7, 5 litres and 250ml, 50 ml for the conventional flow rate measurement. System loss might be happened as the loss volume of wastewater approximately for 100, 50 ml on time basis. The peak hour of the flow rate of the textile vat wastewater was almost 2 pm because of the presents of all facilities in the vat like cotton, dye, water, labours etc supply. The labours of the textile vat wastewater have been accustomed to taking their meal after just 2 pm. That's why; the flow rate has been drastically decreased and after sometime the flow rate has started increasing. This fluctuation has been shown in the figure 1. Finally the Flow rate was very low at just 6 pm before close of the textile vat. It has been known by reviewing from many literature related wastewater generation in Dhaka city, textile vat dye source discharge less amount wastewater comparing to the textile industries situated in Dhaka city because it is hand driven industry. Textile vat has been shown in Figure 1 and Figure 2 respectively.

Figure 2: Textile vat dye at Kumarkhali Upazilla with red color dyeing image

Figure 3: Textile vat dye at Kumarkhali upazilla with black color dyeing image

3.2 Wastewater characterization and treatment of textile dyeing vat

The pH level of the Bulbul textile was found below 9 where the DoE standard was 9 in pH level. All parameters have been shown in the table 1. The Total Suspended Solid was 408mg/L and 400mg/L for the month of June and July which has been exceeded the limit of DoE where the DoE standard of Bangladesh was 100mg/L. The TSS analysis has revealed the excessive limit comparing to the DoE, Standard (TSS standard 100mg/L). Total Dissolved Solid was 2601 mg/L and 2540 mg/L for the month of June and July respectively which has shown that TDS has been exceeded DoE standard (TDS Standard 2100 mg/L). Turbidity of the waste water of textile vat dye was 452.5 NTU and 449.29 NTU for the month of June and July in 2012 respectively that shows the over limit. On the other hand, drain and river waste water shows the turbidity that does not exceed the limit of DoE standard. The range of colour, BOD, COD, Chromium and sulphate of the textile vat dye wastewater were 5170 Pt-Co, 233.508 mg/L, 381 mg/L, 0.2 mg/L and 751.35 mg/L for the month of June 2012 respectively and 5050 Pt-Co, 213.948 mg/L, 372 mg/L, 0.2 mg/L and 750 mg/L for the month of July 2012 respectively. The analysis of Turbidity, Colour , BOD, COD, Chromium and sulphate for all sources like textile vat dye, Drain and River have been shown in the table 1 as well.

Table 1: Wastewater parameters of textile vat dye, Drain and River

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Parameter	Textile vat dye		Drain		River Confluence	
	June	July	June	July	June	July
pH	8.14	8	7.59	7.5	7.42	7.37
Turbidity (NTU)	452.5	449.29	295.03	294	75.2	69.3
Color (Pt-Co)	5170	5050	3210	3190	2410	2360
TDS (mg/L)	2601	2540	747	740	333	320
TSS (mg/L)	408.60	400	303.83	300	62.43	60
BOD_5(mg/L)	233.50	213.94	145.53	140.91	16.5	15.2
COD (mg/L)	381	372	226	224	25	24
SO_4(mg/L)	751.35	750	750	750	67	65
Cr (mg/L)	0.2	0.2	0.15	0.16	0.07	0.07

Numerical values of Influent, Effluent and Removal Efficiency of the textile vat wastewater have been shown in the table 2.Here influent parameter indicates the values of raw textile vat wastewater. Besides the effluent parameters indicates the treated wastewater by Ferric Chloride. The more the TDS value, the more will be the turbidity, colour, BOD_5 and COD values. In table 1, it has been shown that the more the TDS value, the more will be the turbidity, colour, BOD_5 and COD respectively textile vat dye, drain and river confluence. After treatment, TSS has been totally removed, that's why it is possible to reach the removal efficiency for TSS by 100%. Ferric Chloride itselt acted as the coagulant and acid because after preparation of the stock solution with the help of distilled water under the treatment phase, the product proton acted as the deeds of acid whatever it does. By this way, proton has lowered the pH level of the textile vat wastewater.

Here, pH values has been lowered from 8 to 6 and reached 24% removal efficiency. Besides the major parameters of the wastewater of the textile vat like turbidity, colour, COD and BOD_5 have been lowered after the treatment of wastewater and reached the removal efficiency respectively 98.86%, 99.14%, 85.07% and 89.59%. Mahbubul *et* al has showed that all the textile wastewater discharged in Dhaka City has high values of all parameters whereas the textile vat dye situated in Kumarkhali upazilla has been discharging wastewater with low values comparing Dhaka city though textile vat wastewater has been crossed the limit of DoE, Bangladesh. For Treatment, it has been needed less amount of Coagulant, Ferric Chloride. The result has revealed that no satisfactory effect has been shown to lower the level of sulphate. So, the removal efficiency of the sulphate is only 6.795%.

Oil and grease of the textile vat wastewater has not been characterized in this research work. It's because of proper lab facility. Of all the heavy metal, only chromium has been characterized in this research work. Chromium removal by the coagulant chromium has been satisfactory and the removal efficiency of the chromium is 85%. This is the first time research work done in this textile industrial area. For that case, it is not possible to show how results and interpretations agree (or contrast) with previously published work.

Theoretical implication of my research work in a word reveals the jar test method for chemical treatment by ferric chloride. During this process, preparation of the stock solution of ferric chloride, arrangement of the six biker for jar test, fulfilment of the biker with wastewater, variations of dose of stock solution of ferric chloride, one minute rapid mixing and fifteen minutes slow mixing with jar test apparatus, fifteen minute rest and finally characterization of effluent (treated raw textile vat wastewater) are the basis of the theoretical implication of my research work. As the textile vat wastewater by its characterization is less concentrated comparing to the textile wastewater in Dhaka city as well as others, it needs less amount of chemical (ferric chloride) to run Effluent Treatment plant of this area which helps environmental management through wastewater treatment. Thus my research work has indicated pathways for possible practical applications by setting up ETP (Effluent Treatment Plant).

Table 16: Numerical values of Influent, Effluent and Removal Efficiency

parameter	Influent	Effluent	Removal Efficiency
pH	8.07	6.12	24.16
Turbidity (NTU)	450.29	5.1	98.86
Color (Pt-Co)	5100	43.66	99.14
TDS (mg/L)	2570	1700	33.85
TSS (mg/L)	404.33	0	100
COD (mg/L)	375	56	85.07
SO_4(mg/L)	750.67	700	6.79
Cr^{6+}(mg/L)	0.2	0.03	85
BOD_5(mg/L)	221	23	89.59

4. CONCLUSIONS

Characterization of textile wastewater has shown the environmental pollution near the Textile Dyeing Vat. If there will not be setup any ETP further, there will be seen a very vulnerable environmental condition in this study area. Flow dynamics has expressed the perception of the wastewater discharge in volume from a Textile Vat per day. For this work, the Characterization, Flow dynamics and Treatment of the effluent may be acted as the forerunner in setting up Effluent Treatment Plant or Common Effluent Treatment Plant. This research work will help replicating through such research like treatment and management of textile waste water in Kumarkhali Upazilla that will provide the international provisions to export the textile product which improves economic development of the industry, the maintenance of sustainable water quality to enhance the quality of life of human and aquatic organism of the river and opportunity to involve more textile labors as well as insurance of their better livelihood pattern, all of which terms as Sustainability in Management.

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REFERENCES

Guha, A.K.(2009). Characterization of textile wastewater of different areas of Bangladesh, Bangladesh Textile Today, 3(1), 28-31.

Lin, S.H. and Chem, M.L.(1997).Treatment of textile waste water by chemical methods for reuse. Water Res 31:868-876, India.

Mahbubul, H.(2008). Treatment of Textile waste water in Bangladesh, cotton Bangladesh.

Kulkaria, B.V.(2007). Sequential coagulation studies for primary treatment of textile process effluent instead of acid neutralization. Journal of the industrial pollution control, 23(1), 123-130.

Rahman. M.M. and Shaid A.(2009). Semi Biological Effluent treatment Plant in Textile Waste Water Management. International Journal of Engineering and Technology 6(1), 404-409.

MEDICAL WASTE MANAGEMENT AT HOSPITALS IN DHAKA CITY: A CASE STUDY

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ABSTRACT

Medical Waste (MW) is infectious and hazardous. It poses serious threats to environmental health and it requires specific treatment and management prior to its final disposal. Improper disposal of MW provides hazardous situation which leads to contamination of nearby communities. This research focuses on the present status of Medical Waste Management (MWM) in Dhaka city and to assess the possible environmental consequences of improper disposal of MW. Two hospitals in Dhaka were purposively selected as sample for data collection to quantify the amount of wastes generated from here and observing the existing system of MWM. It was observed that the non-hazardous waste was treated and disposed with the domestic waste of Dhaka City Corporation (DCC) and hazardous waste was managed specially. The amount of total generated hazardous waste of Shaheed Sohrawardy Medical College Hospital (SSMCH) was found 15.02 kg/day, 23.84 kg/day, and 15.425 kg/day in 2010, 2011, and 2012 respectively. From National Institute of Mental Health (NIMH) generated waste amount was 3.02 kg/day, 4.28 kg/day and8.07 kg/day in 2010, 2011, 2012 respectively. But it has been observed existing MWM is not satisfactory at all. The work also reveals that the level of knowledge and awareness about the risk of MW among the individuals involved in the medical industry needs to be risen to standard level to ensure spontaneous participation to proper management of MW. Also there is a need for an improvement of handling and disposal methods of MW in almost all the available Health Care Facilities (HCF).

Keywords: Dhaka City Corporation, Medical Waste, Improper disposal, Waste Management.

1. INTRODUCTION

MW may carry germs of dreadful diseases. So that simply disposing of MW in dustbins, drains and canal or finally dumping it to the outskirts of the city possess a serious public health hazard (PRISM, 2005). The problem is getting worse due to the expansion of medical facilities in the world. The environmentalists are becoming very much worried because of the detrimental effect of MW. As a result MW has become a major issue in urban areas of countries of the world as it spreads diseases at a faster rate than others. In hospitals, different kind of therapeutic procedures are carried out and result in the production of infectious waste, sharp objects, radioactive waste and chemical material (WHO, 2002). According to WHO (BAN & HCWH, 1999) approximately 85% or hospital wastes are actually non-hazardous, 10% are infectious, and around 5% are non-infectious but hazardous. In the US for example, about 15% of hospital waste are regulated as infectious waste. In India this could range from 15 to 35% depending on the total amount of waste generated (BAN & HCWH, 1999). In Pakistan about 20% of hospital waste is found to be potentially infectious or hazardous (Agarwal, 1998). The total garbage generation in Dhaka city is 3500 metric ton per day from which only 5.7 % comes from medical establishment (Asaduzzaman & Hye, 1997). Around 200 metric tons of hospital wastes are generated per day in the city of Dhaka, Bangladesh. Of this amount, roughly 20% is infectious and hazardous (Kazi, 1999).

In Bangladesh, MW account for a very small fraction, about 1% of the total solid wastes generated in all over the country (World Bank Health Facility, 2002). However, when this tiny amount is not managed properly and mixed with other domestic waste and then whole waste stream becomes potentially hazardous. Bangladesh, especially the capital city of Dhaka, is facing the impact of urbanization as well as the increase in HCF. The Dhaka city was established in 1608 along the bank of the Buriganga River. The metropolitan city of Dhaka now has an area of 360 sq. km. The climate is tropical with heavy rains in the monsoon season and bright sunshine for most of the year. The rapid rise in population of Dhaka City has been caused mainly by the large number of people migrating from rural areas. The rapid increase of hospitals, clinics, diagnostic laboratories etc in Dhaka city exerts a tremendous impact on human health ecology. More than 600 clinics and hospitals exist in the DCC. These facilities generate an estimated 200 tons of waste a day (Lawson, 2003). At present more than 2000 HCFs are existing in the DCC (DCC Report: 2012). But only a few have the necessary means to dispose the waste safely. Untreated MW is posing a serious health hazard to city dwellers. This has resulted in a decline in sanitation, which in turn causes adverse health impacts. The legal framework is not supported by timely enforcement actions and there is a general lack of funding to develop common facilities for efficient MWM It is reported that even body parts are dumped on the streets by these HCE. The present practice of improper handling of generated hospital wastes in Dhaka city is playing a contributing role in spreading out the Hepatitis and HIV diseases. The liquid and solid wastes containing hazardous materials are simply dumped into the nearest drain or garbage heap respectively where they are prone to contaminate the rag-pickers that sift through the garbage dumps. The chances of infection are very high to the cleaners, concerned people in the HCE and to the general population. The improvement of waste management for the HCE in Bangladesh will have significant long-term impact on keeping the spread of infectious diseases to a minimum and result in a cleaner and healthy environment. The objective of this paper is to quantify the amount of wastes generated by two selected HCF. In addition, the paper presents the current waste handling practices in terms of storage, collection, transportation and disposal within and outside hospital premises in Dhaka City and also to assess the possible environmental consequences of improper disposal of MW.

A Recent Past Scenario of MWM in Dhaka city

Up to December 2005, there has been an improper procedure of MWM in Dhaka City. No HCF segregated their generated wastes, except a very few. But MW needs to be segregated separately according to their characteristics at the point of generation. In some HCF, all the infectious wastes were found to be separated from the non-infectious waste stream at the site of production, but during disposal in the DCC dustbins the wastes were then mixed together. The figure 1 shows the previous medical waste management practices in Dhaka.

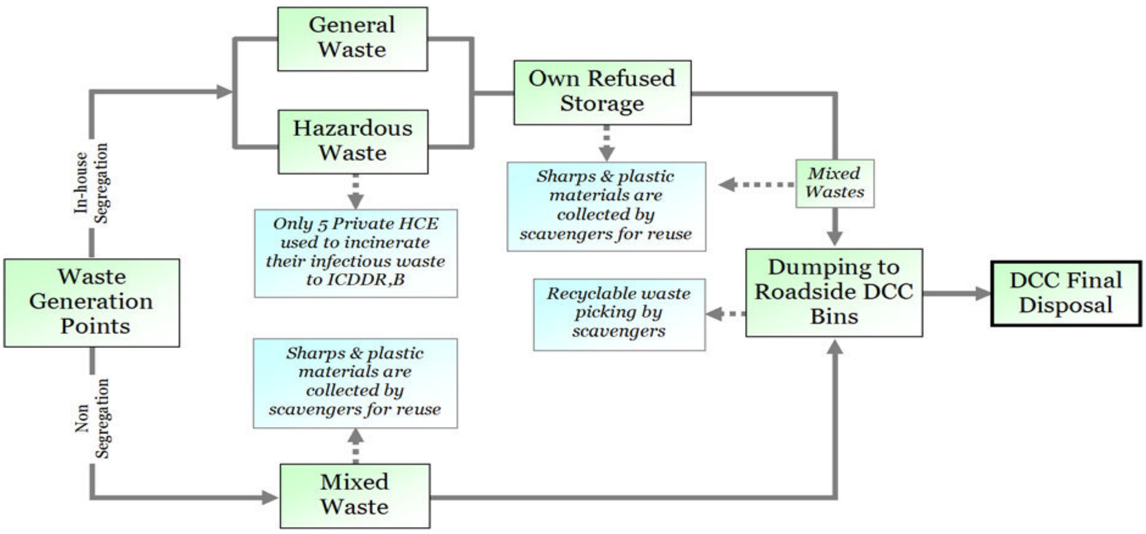

Figure 1 Previous medical waste management practices in Dhaka City

New Approach in Generation, discharge, collection, transport and disposal of MW in Dhaka City:

Medical facilities in different HCFs in Bangladesh are characterized by inadequate and inappropriate refuse storage facilities, lack of refuse collection services, improper disposal methods and inadequate and inappropriate protective gear for refuse handlers. In Bangladesh, proper medical waste management is a new phenomenon and Government of Bangladesh (GoB)] is trying to develop a new and modern approach to deal with the medical waste properly.

PRISM (Project in Agriculture, Rural Industry, Science and Medicine) Bangladesh, a reputed national NGO in Bangladesh, is now working for MWM in association with the DCC. With financial and technical support from Water and Sanitation Program (WSP), PRISM Bangladesh carried out a survey on the MWM in Dhaka City. Subsequently, PRISM Bangladesh with the financial support from Canadian International Development Agency

(CIDA) has recently developed a disposal facility for low cost medical waste treatment and management in Dhaka City. The DCC has allocated one acre (0.405 hectare) of land in Matuail, a dumpsite near the city limit for the final disposal of medical waste. It is inadequate to handle all the MW of the city with the limited facilities of final disposal. PRISM Bangladesh is managing the generated medical waste in different forms (Figure 2)

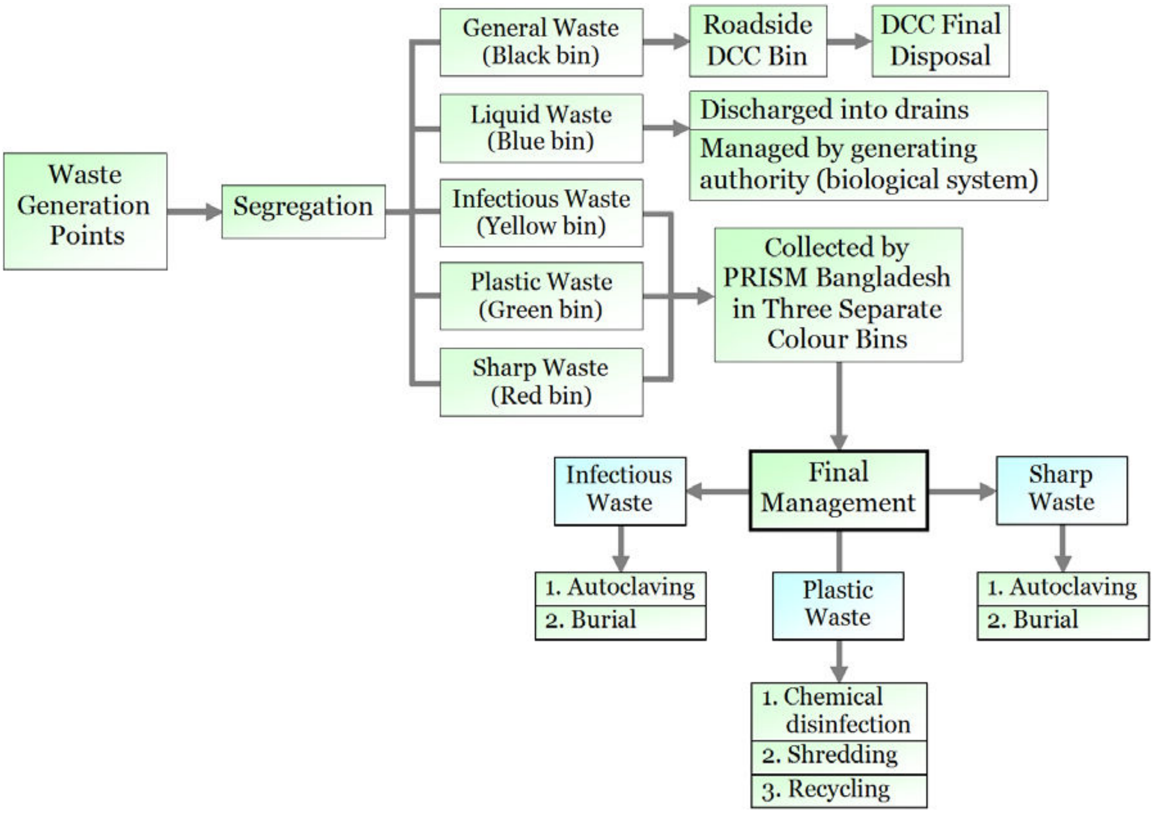

Figure 2 Existing MWM practice in some HCF Dhaka City organized by PRISM Bangladesh. This new system has been practiced since December 2005.

In the new approach, PRISM Bangladesh is involved in training relevant personnel of different HCF for increasing awareness and proper in-house management of MW. PRISM Bangladesh has developed a system for collecting segregated hazardous wastes (except radioactive wastes) from each HCF through newly set up vehicles to carry this waste for final dumping at their newly developed Matuail Plant.

It has introduced in-house storage of MW in color-coded bins approved by GoB. Segregation of waste at source into suitable color-coded bins is vital to a proper waste management. The color codes recently introduced by the GoB should be followed at all HCF for this purpose. Infectious waste should be packaged. The integrity of packaging can be preserved during handling, storage, transportation and treatment. The color-coded waste segregation guide represents best practice and ensures, at minimum, compliance with current regulations. Some HCF in Dhaka City are paying a service charge for the collection from their premises and final management at the Matuail plant.

Pictorial view of existing Medical Waste Management in Dhaka

MW Source Collections

Safe Transportation of MW

Incineration of MW

Chemical Disinfection of MW

Autoclaving of MW

Sharp MW Disposal

Figure 3 Medical waste collection and treatment facility in Dhaka city

Potential Risk of Untreated MW

Medical waste contains highly toxic chemicals, pathogenic viruses and bacteria, which can lead to pathological dysfunction of the human body. MW presents a high health risk to doctors, nurses, technicians, sweepers, hospital visitors and patients due to arbitrary management. It is a common observation in Dhaka City that poor scavengers, women and children collect some of the MW (e.g. syringe-needles, saline bags, blood bags etc.) for reselling despite the deadly health risks. It has long been known that the re-use of syringes can cause the spread of infections such as AIDS and hepatitis. The collection of disposable medical items (particularly syringes), its re-sale and potential re-use without sterilization could cause a serious disease burden. The safe disposal and subsequent destruction of medical waste is a key step in the reduction of illness or injury through contact with this potentially hazardous material, and in the prevention of environmental contamination. . Many field survey shows that almost all the HCF discharge their liquid pharmaceutical and chemical waste into the general sewers or drains in Dhaka City because none of them have any proper liquid waste management facilities. Liquid waste is mainly generated from patients' service units, operation and surgical units, laboratories and other health-care units.

Persons at Risk

All individuals exposed to hazardous health-care waste are potentially at risk, including those within health-care establishments that generate hazardous waste and those outside these sources who either handle such waste or are exposed to it as a consequence of careless management. The main groups at risk are the following:

• Medical doctors, nurses, health-care auxiliaries and hospital maintenance personnel
• Patients in health-care establishments or receiving home care
• Visitors to health-care establishments
• Workers in support services allied to health care establishments such as laundries, waste handling and transportation
• Workers in waste disposal facilities (such as landfills or incinerators), including scavengers

The hazards associated with scattered, small sources of health care waste should not be over looked; waste from these sources includes that generated by home-based health- care, such as dialysis and that generated by illicit drug use.

Figure 4 Persons at risk-Workers in waste disposal facilities

Impacts of Infectious and sharp waste

For serious virus infections such as HIV/AIDS and hepatitis B and C, hospital workers; particularly nurses-are at greatest risk of infection through injuries from contaminated sharps (largely hypodermic needles). Other hospital workers and waste management to operators outside hospitals are also at significant risk, as are individuals who scavenge on waste disposal sites (although these risks are not well documented). The risk of this type infection among patients and the public is much lower. Certain infections spread through other media or caused by more resilient agents may pose a significant risk to the general public and to hospital patients. Individual cases of accidents and subsequent infections caused by the hospital waste are well documented. For example; a hospital housekeeper in the USA developed staphylococcus bacteria endocarditic after a needle injury.

Possible Environmental Consequences of Improper Disposal of MW:

• Pollutants from MW (e.g. heavy metals and PCBs) are persistent in the environment.

• Accumulation of toxic chemicals within soil. (Proximity to agricultural fields, humans, soil Organisms, wildlife, cattle)

• Ground water contamination, decrease in water quality.

• Bio-accumulation in organism's fat tissues, and biomagnified through the food chain.

• Repeated and indiscriminate application of chemicals over a long period of time has serious adverse effects on soil microbial population reducing the rate of decomposition, and generally lowering the soil fertility.

• Pathogens lead to long term accumulation of toxic substances in the soil.

• Causes disease and illness in man, either through direct contact or indirectly by contamination of soil, groundwater, surface water, and air.

• Windblown dusts from indiscriminately dumping also have the potential to carry hazardous particulates.

• With domestic animals being allowed to graze in open dumps, there is the added risk of reintroducing pathogenic micro-organisms into the food chain.

• Public nuisance e.g. odors, scenic view, block the walkway, aesthetics, etc.

• Improper sterilization of instruments used in labor room may cause infection to mother and child.

• Combination of both degradable and non-degradable waste increase the rate of habitat destruction due to the increasing number of sites necessary for disposal of wastes

• Plastic-bags, plastic containers, if not properly destroyed may contaminate the soil and also reduces the chance for water percolation into the soil during precipitation

• Open air burning does not guarantee proper incineration, and releases toxic fumes (dioxin) into the atmosphere from the burning of plastics i.e., PCB's. (Nasima, 2000)

2. RESULTS AND DISCUSSION

In Shaheed Sohrawardy Medical College Hospital (SSMCH) and in National Institute of Mental Health (NIMH),there were no proper MWM before 2010. Even MW was just categorized by infectious and non-infectious waste. Non-infectious waste were treated with other domestic waste of DCC and all infectious were collected separately. Then in 2011 the authority became little bit aware to segregate their MW into different color coded bin during its storage in hospitals.

Shaheed Sohrawardy Medical College Hospital (SSMCH)

It is one of the biggest Governments hospitals in Dhaka city. It was established in 1964. It is situated at Sher-e-Bangla Nagor, Dhaka-1207. SSMCH contains 375 no of beds. Very soon it will provide 850 bed facilities for patients. Now it already provides medical facilities for about 350 resident patients and about 600-700 out patients every day and produces a large amount of MW. The quantity of hazardous waste generates from this hospital is approximately 15-25 kilograms of hazardous waste per day.

National Institute of Mental Health (NIMH)

It is also a government owned hospital in Dhaka city. It was founded in 1981 and situated at Sher-e-Bangla Nagor, Dhaka-1207. The NIMH provides medical facilities for about 150 resident patients and about 100-150 out patients every day and produces a medium quantified amount of MW. The amount of hazardous waste generates from this hospital is approximately 5-10 kilograms of hazardous waste per day.

Table 1 Details of the two selected hospitals:

Name	Bed capacity (Number)	Patients (Number)	Department (Number)
SSMCH	375	350	32
NIMH	200	150	07

Present situation of MWM at two hospitals:

Collection and Storage

Collection and storage of waste in SSMCH and NIMH was quite systematically. The generated waste from these hospitals was segregated into general, sharp, plastic, liquid and infectious type at the point of generation. Then wastes were collected and stored into separate color coded bin.

Table 2 Details of color coded container for MW storage approved by GoB:

Types of waste	Material of containers	Color of containers
Non infectious	Plastic	Black
Infectious	Plastic	Yellow
Plastic or recyclable	Plastic	Green
Sharp	Plastic	Red
Liquid	Plastic	Blue

Though GoB has already approved 5 colored bins for 5 categorized MW, but DCC only provides 3 colored bins- black for non infectious waste, yellow for infectious waste and green for plastic or recyclable waste and sharp waste in these hospitals. Then sharp waste was separated lastly at the disposal site before final disposal. It was observed that liquid waste was not collected in NIMH where as in SSMCH it was collected.

Figure 5 Color coded container used in hospitals

Transportation

The non-hazardous waste was collected and disposed with other domestic waste of DCC. But the hazardous waste was collected in every morning by the staff of the PRISM, Bangladesh. Then the waste was transported to the final disposal sites, Matuail.

Figure 6 Transport used by PRISM to transfer the MW from hospitals to Matuail

Processing, Recovery and Final Disposal

After transfer and transportation of MW to Matuail, among all the wastes collected from hospitals recyclable wastes were processed for material recovery. At first manual sorting was done and then recyclable waste, such as plastics, glass etc. were disinfected chemically. After that the disinfected wastes were dried out into some open concrete chamber constructed in Matuail thus shown in figure 7 and then shredded into powder form through shredder machine. Other infectious wastes were disposed by incineration, autoclaving and deep burial process according to its condition.

Figure 7 Processing of Material Recovery in Matuail

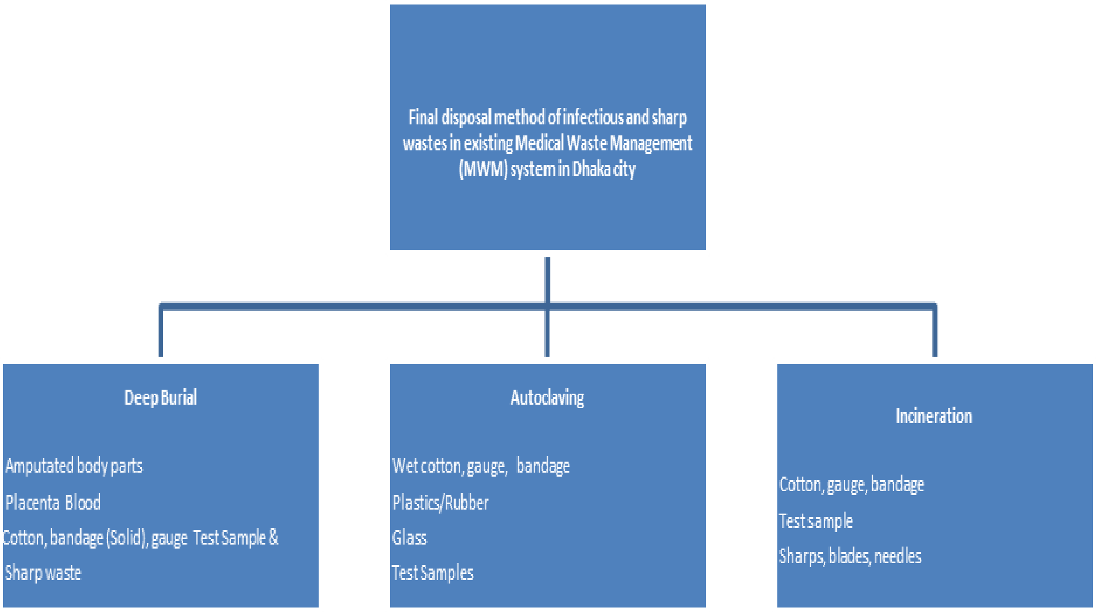

Figure 8 Flow diagram of existing disposal method in Matuail

Data Analysis and observation

The composition and generation rate of MW varies in two HCF according to their infra structural facilities to handle the no of patients and types of diseases. Data was supposed to be collected on a daily basis and represented by tabular and graphical form below here:

Table 3 Quantity of categorized MW collected from SSMCH

Year	Infectious Waste (kg/day)	Plastic waste (kg/day)	Sharp waste (kg/day)	Liquid waste (kg/day)	Total waste (kg/day)
2012	9.65	1.175	2.55	2.05	15.425
2011	9.90	1.96	5.1	6.88	23.84
2010	13.40	-	1.62	-	15.02

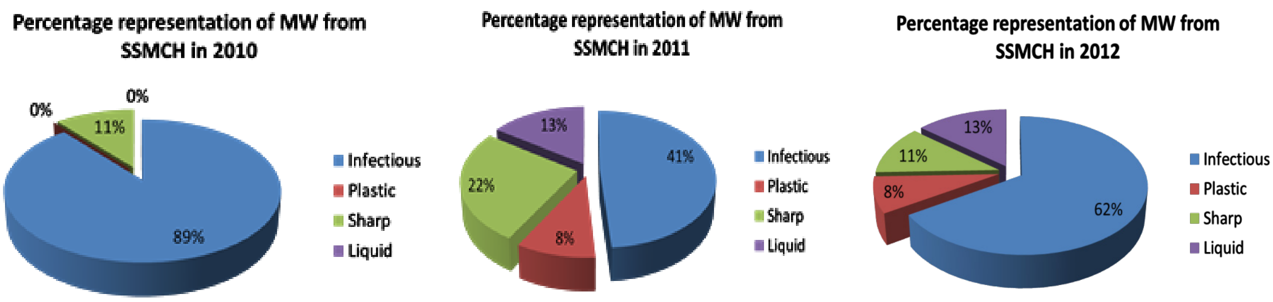

Figure 9 Composition of MW in SSMCH in 2010, 2011 and 2012

Table 4 Quantity of categorized MW collected from NIMH

Year	Infectious waste (kg/day)	Plastic waste (kg/day)	Sharp waste (kg/day)	Liquid waste (kg/day)	Total waste (kg/day)
2012	4.986	1.58	1.5	-	8.07
2011	2.80	1.38	.27	-	4.45
2010	2.52	.23	.52	-	3.26

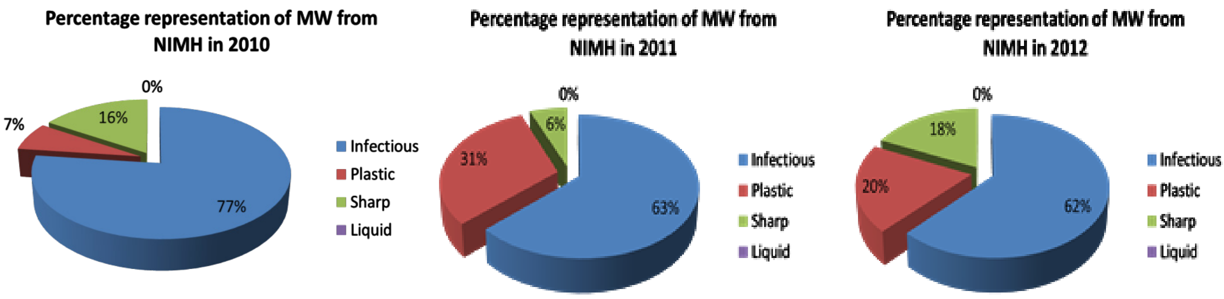

Figure 10 Composition of MW in NIMH in 2010, 2011 and 2012

Generally, total amount of MW increases with time. But it has been found that in 2012, total MW is less than that of 2011 in SSMCH. It can be happened as waste generation may vary with season. Also it is noticeable that liquid waste was not collected in NIMH. It is very common phenomenon that many medical staffs collect syringe-needles, saline bags, blood bags etc. for reselling despite the deadly health risks, thus total waste can decrease. Also record keeping system was not fully reliable.

3. CONCLUSIONS

Proper management of medical waste is crucial to minimize health risks. The improvement of present waste management practices for HCF in Bangladesh will have a significant long-term impact on minimizing the spread of infectious diseases. Such type of study is essential because data regarding the true pattern of medical waste management is not identified in our country. As the study is not based on a representative sample size it may need replication so that findings become established. It has been found that a high rate of medical waste mismanagement is happening in Dhaka city which will create adverse effect on health. Also it was observed that MWM in SSMCH & NIMH was not up to the mark at all. Above all findings of this study clearly indicate that proper attention is required for MWM by regarding authority otherwise it will create adverse effect on health.

REFERENCES

Agarwal, R. (1998). “Medical waste disposal issue, practice and policy, An Indian and International Perspective. Seminar on health and environment”, Centre for Science and Environment, New Delhi, India.

Asaduzzaman, M., & Hye, S.A.(1997). “When both market and state fail: The crisis of solid waste management in urban Bangladesh”.

BAN & HCWH. (1999). “Medical Waste in Developing Countries, An analysis with a case study of India, and A critique of the Basel -TWG guidelines. Basel Action Network (BAN) secretariat, Asia Pacific Environmental Exchange, Seattle, USA.

Dhaka City Corporation,(2012) . Personal communication.

Governance. A review of Bangladesh’s Development. (1997). “Centre for policy dialogue. University Press30 Limited”, Dhaka, Bangladesh.

Haque, (1994). “Waste Ladfill and Hospital Waste Incineration.Roteb-Solid Waste Consultency”, Kleinpolderplein , Rotterdam, The Netherlands.

Kaji, N.M. and Chowdhury,A.M.R.(1999). “ Environmental Investigation of Medical Waste Management System in Bangladesh with Special References to Dhaka City”, BRAC, Research and Evaluation Division.

Lawson A. (2003). “UN tackles Dhaka's medical waste” ,Dhaka.

Nasima, A. (2000). “ Medical Waste Management: A Review, Environmental Engineering Program, School of Environmental resources and development” , Asian Institute of Technology, khlogluang, Pathumthani, Thailand.

Rokunuzzaman, M et al.(2009) “Medical Waste Management in Some Selected Metropolitan Cities of Bangladesh”, presented at an International conference on Solid Waste Management in Khulna, Bangladesh.

PRISM Bangladesh .(2005) “Survey report of hospital waste management in Dhaka city”.

WHO (2002) Waste from Health Care Activities, World Health Organization.

SPECIATION OF COPPER IN WATER AND POTENTIAL OF APPLICATION OF COPPER SALT FOR ALGAE REMOVAL AT THE SAIDABAD WATER TREATMENT PLANT

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ABSTRACT

The Saidabad Water Treatment Plant (SWTP) serves as a major potable drinking water source for the Dhaka city which faces high concentration of algae problem during dry season. This study evaluates the possible application of copper salt in controlling algae at the SWTP. In this study, the chemical speciation of copper has been assessed using the software MINEQL+ considering major ions present in the raw water of SWTP (e.g., Cl^-, NH_3-NH_4^+, NO_3^-, PO_4^{3-}, SO_4^{2-}, HCO_3^{2-}, Ca^{2+}). Copper speciation has been found to be primarily a function of pH, alkalinity and initial copper concentration. Under the water quality conditions considered, the maximum concentration of dissolved copper achievable at pH 7 has been found to be too low compared to the requirement for algae control, which is 0.5mg/l. However, lower pH would promote higher concentration of dissolved and free copper. Removal of residual copper from water is an important issue after addition of copper salt. Since SWTP uses alum coagulation as a treatment process, removal of copper by alum coagulation was assessed. It has been found that copper removal is a strong function of pH, with removal increasing rapidly with increasing pH; copper removal efficiency did not change significantly with alum dose considered in this study (25 to 50 mg/l).

Keywords: Abstract, Objective, Methodology, Results, Conclusions

1. INTRODUCTION

Copper (Cu) is plentiful in the environment and essential for the normal growth and metabolism of all living organisms (Schroeder et al., 1966; Carbonell and Tarazona, 1994). Copper, if present in water in excess of allowable limit, is not only harmful to human but also for aquatic environment (USEPA, 1980). The Bangladesh drinking water standard for copper is 1.0 mg/l; while the effluent discharge standard (to inland surface water) is 0.5 mg/l. In fact, copper is one of the most toxic metals to aquatic organisms and ecosystems. Because of its toxic characteristics, copper is often used for controlling the growth of algae in water. However, the toxicity of copper depends on its speciation, i.e., chemical forms in water, which in turn depends on a number of factors including pH, Alkalinity, initial copper concentration and presence of other ions. The overall objective of the present research was to assess the chemical speciation of copper in water under different water quality scenarios and to evaluate the effectiveness of copper in controlling algae problem at the Saidabad Water Treatment Plant (SWTP), which suffers from high concentration of algae and ammonia during the dry season.

2. METHODOLOGY

In the present research work, the chemical speciation of copper in water has been assessed using a software named MINEQL+ version 4.6 . The chemical speciation carried out in this study focuses on the effect of pH, alkalinity and the concentrations of the initial copper ions. Three different water quality scenarios were considered for the simulation. At first speciation of copper was simulated considering all major ions present in the natural water. Here the effect of Cl^-, NH_3-NH_4^+, NO_3^-, PO_4^{3-}, SO_4^{2-}, CO_3^{2-}, Ca^{2+}, pH and initial copper concentration were considered. Next analysis was done considering only the major components (pH, alkalinity, initial copper concentration) that were found to control the chemical speciation of copper in water. Finally for assessing the chemical speciation of copper, the water quality of the Saidabad Water Treatment Plant (SWTP) has been considered, in an effort to assess the effectiveness of possible application of Cu salt for controlling high algae in the raw water at the SWTP. In addition, the removal of Cu from water by adsorption using alum coagulation has also been assessed through laboratory batch experiments. The laboratory batch experiments have been carried out to understand the effects of pH and alum dose on the adsorption/ removal characteristics of copper. Alum coagulation has been used in this study because this process is also used at the SWTP.

2.1 Description of MINEQL+

2.1.1 Principal of analysis in MINEQL+

MINEQL+ performs analysis using chemical equilibrium method. Chemical constituents that are dissolved in water may form chemical complexes, precipitates as solid phases, de-gas from system or absorb onto particulate surfaces. All of these reaction pathways are affected by, and will affect, water quality parameters such as pH, alkalinity or ionic strength. The chemical equilibrium approach offers a way to understand these chemical interactions in a straight forward, unified manner (Brown and Allison, 1987).

MINEQL+ allows one to investigate the speciation of any type of aqueous chemical system. The approach of solving chemical equilibrium problems can be broken down into 5 simple steps:

- Selection of chemical components that will define the system;
- Creation of chemical species from these components (this step may include scanning a database of thermodynamic constants or creating new species);
- Setting the total concentration of individual components;
- Running the calculation (this step may include setting ionic strength, temperature or surface complexation options);
- Viewing and extracting output data (this step may include creating graphics plots, specialized reports or saving data as files for use in other software).

2.1.2 Water quality simulation using MINEQL+

MINEQL+ version 4.6 has been used for the simulation of water quality; speciation of copper and other species were simulated for different scenarios. At first, simulation was run with all major constituents present in natural water and speciation of copper was analyzed. The purpose of these simulations was to assess which constituent has the most influence on copper speciation. The composition of water used in these simulations was set considering the raw water quality at the SWTP. In the simulations, only value of one parameter was varied at a time, while concentrations of other parameters were kept at their base values (Table 1). At the end of each simulation, speciation of copper was analyzed. The copper species (including complexes and solids) considered in the simulations included the following:

Cu^{2+}; $Cu_2(OH)^{2+}$; $Cu(OH)^{3-}$; $Cu(OH)_4^{2-}$; $CuOH^+$; $Cu(OH)_2$ (aq); $CuHCO_3^+$; $CuNH_3^{2+}$; $CuCl_3^-$; $CuCl_2$ (aq); $CuCl_4^{2-}$; $CuCl^+$; $CuCO_3$ (aq); $Cu(CO_3)_2^{2-}$; $Cu(NO_3)^2$ (aq); $CuNO_3^+$; $CuSO_4$ (aq); CuO (s) (Tenorite); $Cu_2CO_3(OH)_2$ (s) (Malachite); $Cu_2Cl(OH)_3$ (s) (Atacamite); $Cu_3(CO_3)_2(OH)_2$ (s) (Azurite); $Cu_3(SO_4)(OH)_4$ (s) (Antlerite); $CuSO_4.3Cu(OH)_2$ (s) (Brochantite); $Cu_4(SO_4)(OH)_6.2H_2O$ (s) (Langite); $CuSO_4.5H_2O$ (s) (Chalcanthite); Cu_2OCl_2 (s) (Melanothallite); $Cu(OH)_2$ (s); $Cu_2(OH)_3NO_3$ (s); $CuOCuSO_4$ (s); $Cu_3(PO_4)_2.3H_2O$ (s); $CuCO_3$ (s); $Cu_3(PO_4)_2$ (s) and $CuSO_4$ (s).

From a series of run it is identified that only alkalinity, pH and initial Cu concentration significantly influence the speciation of Copper in water. So subsequent analysis was carried out by considering variation of total Cu, alkalinity and pH; and while other parameters were kept fixed at their base values (Table 1). The dissolved species and solids considered in the second stage of simulation include the following:

Dissolved species: $Cu_2(OH)^{2+}$; $Cu(OH)^{3-}$; $Cu(OH)_4^{2-}$; $CuOH^+$; $Cu(OH)_2$ (aq); $CuHCO_3^+$; $CuCO_3$ (aq); $Cu(CO_3)_2^{2-}$; $CuSO_4$ (aq)

Solids: CuO (s) (Tenorite); $Cu_2CO_3(OH)_2$ (s) (Malachite); $Cu_3(CO_3)_2(OH)_2$ (s) (Azurite); $Cu(OH)_2$ (s); $Cu_3(PO_4)_2.3H_2O$ (s); $CuCO_3$ (s) and $Cu_3(PO_4)_2$ (s)

The concentrations of major parameters used in the second stage of analysis are shown in Table 2. The concentration of free Copper, dissolved Copper, major species of Copper present in the water and the precipitated solids were recorded from the output of MINEQL+.

In the third stage of analysis, the water quality of SWTP was considered. Simulations were done to evaluate speciation of copper if copper salt is added to the raw water at the STWP for controlling algae growth. Since only dissolved Cu species are effective in controlling algae growth, the simulations were done to see dissolved copper concentration under different scenarios. Here, fixed values of alkalinity and phosphate, representing raw water at the SWTP, were used in the simulations; while pH and total copper concentration were varied.

2.2 Batch Adsorption Experiments

A major objective of this study was to assess if Cu salts could be used for controlling the growth of algae in the raw water at SWTP. However, if copper salt is added to water, then amount of total copper in the treated water should be within the acceptable limit. Since SWTP uses alum coagulation as a treatment process, removal of copper from water by alum coagulation was evaluated by the batch experiment. All batch experiments were carried out at an ionic strength of 0.01 M NaCl. De-ionized water was used to prepare stock solutions of NaCl and Copper (using $CuSO_4$). Glass volumetric flasks and reaction vessels were cleaned with 10% HNO_3 and then rinsed several times with de-ionized water before use. For batch experiments, 500 mL of de-ionized water was taken in a set of glass beakers. Then required amount to NaCl stock solution was added to the beakers to raise the ionic strength to 0.01 M. Then required amount of copper stock solution was added so that concentration of Cu in each beaker is 1 mg/L. Biological buffer (MES) was used to keep the pH constant in a beaker. Two sets of experiments were carried out; one with an alum dose of 50 mg/L and the other with an alum dose of 25 mg/L. In first set, pH values in the 7 beakers were fixed at 4.0, 4.5, 5.0, 5.5, 6.0, 6.5 and 7.0.In second set, pH values in the 6 beakers were fixed at 4.0, 5.0, 5.5, 6.0, 6.5 and 7.0 respectively. A jar test apparatus was used for the batch coagulation experiments. After addition of required alum dose (from alum stock solution), coagulation was done by 1 minute of rapid mixing at 45 rpm and 14 minutes of slow mixing at 25 rpm. The contents of the beakers were then allowed to settle for 30 minutes. The pH of water in each beaker was then measured with a pH meter. Then water samples were collected from the top of each beaker with a pipette for analysis of copper concentration using an Atomic Absorption Spectrophotometer (AAS; Shimadzu AA6800).

3. RESULTS AND DISCUSSION

3.1 Reactions Controlling Copper Speciation

From results of simulation, it was found that Cu speciation mainly depends on Alkalinity and pH. Other common ions present in water have no significant effect on the speciation. So subsequent simulations were carried out by varying pH (5.0 to 9.0), and Alkalinity (10 to 600 mg/L); initial copper concentration was varied from 0.05 to 1.5 mg/L. At lower pH value, no copper precipitates as solids; at pH 5.0 and alkalinity 200 mg/L, all the copper present in water remains in dissolves forms, though the percentage of free copper (Cu^{2+}) is less (73%) due to the formation of $CuHCO_3^+$ and aqueous $CuCO_3$ (Figure 1). At higher pH value, even more coppers precipitates; at pH 9.0, almost all copper (99.9%) is precipitated as solids (Tenorite) and no dissolved and free copper (Cu^{2+}) is found (Figure 2).

Table 10: Water Quality Parameter and their Simulation Range n the First Stage of Analysis

Parameter	Base Value	Simulation Range
pH	7.00	5.00 to 9.00
Cl^-	40 mg/L	40 mg/L
NH_3	10 mg/L	6 to 10 mg/L
NO_3	0.20 mg/L	0.20 mg/L
PO_4^{3-}	2 mg/L	1.2 to 2.8 mg/L
SO_4^{2-}	50 mg/L	5 to 90 mg/L
Cu^{2+}	1.5 mg/L	0.05 to 10 mg/L
Alkalinity as $CaCO_3$	200 mg/L	10 to 600 mg/L

Table 11: Influencing Water Quality Parameters and their Simulation Range in the Second Stage of Analysis

Parameter	Base Value	Simulation Range
Cu^{2+}	1.50	0.05 to 1.50 mg/L
Alkalinity as $CaCO_3$	400	10 to 600 mg/L
pH	7.00	5.00 to 9.00

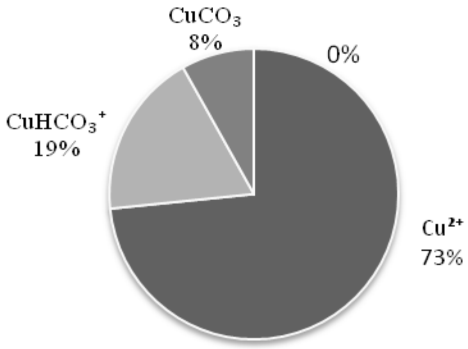


Figure 1: Relative percentage of copper species at pH 5.00, alkalinity 200 mg/l and total copper 1.50 mg/l

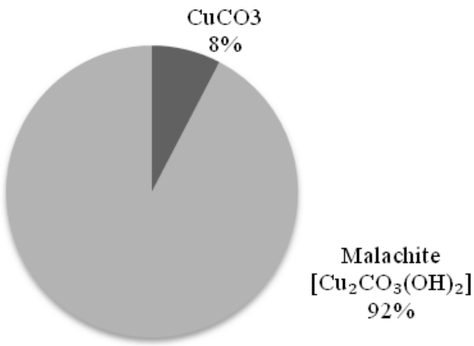
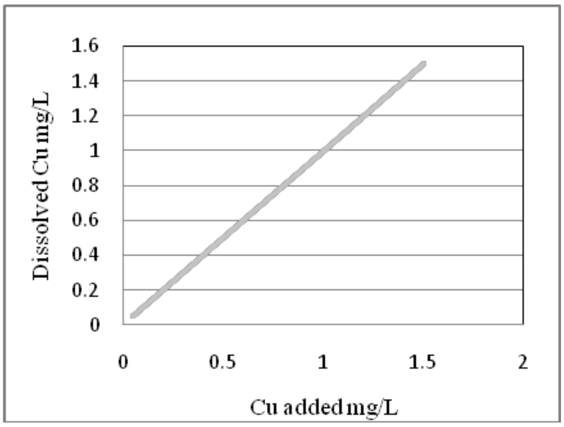


Figure 2: Relative percentage of copper species at pH 7.00, alkalinity 200 mg/l and total copper 1.50 mg/l

3.2 Effect of pH, Alkalinity and Initial Copper Concentration

Simulation of Copper speciation in water shows that copper could be present in water as free cupric ion, complexed cupric ion and precipitated copper solids. In relatively clean water containing common ions present in natural water, copper speciation is a function of alkalinity (i.e., carbonate/ bicarbonate) and pH only. For a fixed pH and fixed initial copper concentration, this speciation varies as a function of alkalinity. Here speciation of copper has been simulated by fixing one component while other components were varied. For a fixed value of pH, alkalinity and copper concentrations are varied to understand its behaviour. The analysis is done for pH 5.00, 6.00, 7.00, 8.00 and 9.00. Alkalinity was varied from 10 to 600 mg/L and copper concentration was varied from 0.05 to 1.5 mg/L. At low concentration of Cu (at or below 0.05 mg/l), and acidic pH values (≤pH 7), no solids of copper forms and all copper exists as dissolved copper. But the percentage of free copper decreases as the concentration of initial copper and pH increases due to the formation of dissolved complexes. Other species present are $CuHCO_3^+$, $CuOH^+$ and $CuCO_3$ at different concentrations. (Figure 3, 4, 5)



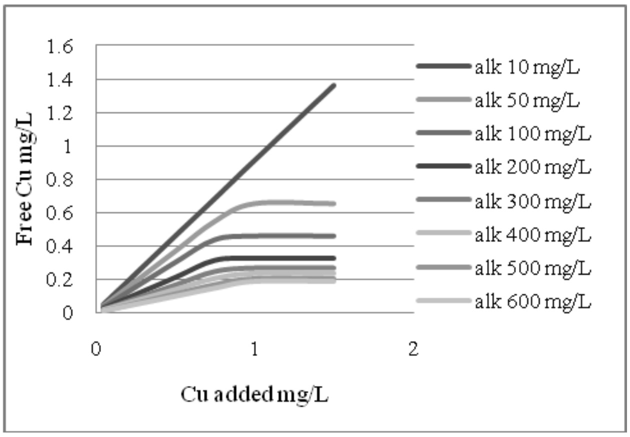


Figure 3: Relationship of initial copper with dissolved and free copper at pH≤6.00 and alkalinity 10 to 600 mg/l

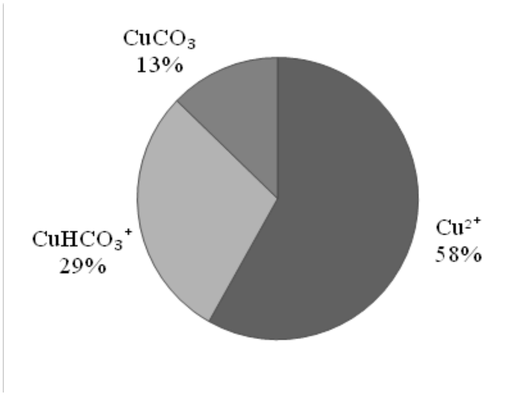


Figure 4: Different copper species at pH 5.00, alkalinity 400 mg/L and total copper 1.5 mg/l

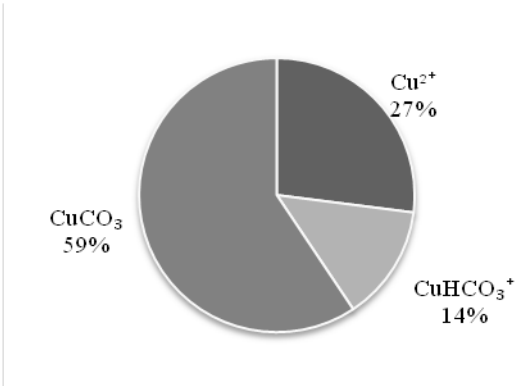
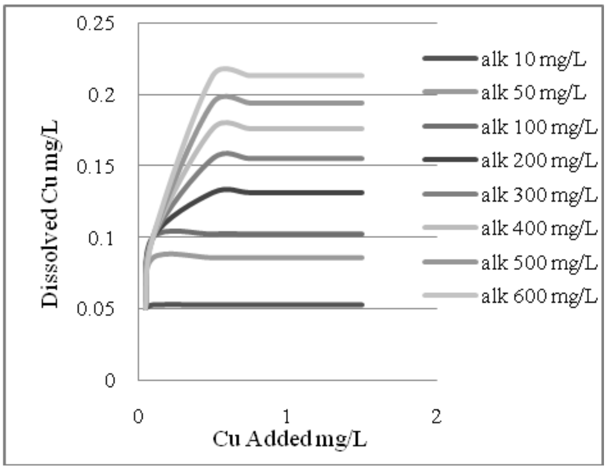


Figure 5: Different copper species at pH 6.00, alkalinity 200 mg/L and total copper 0.75 mg/l

Figure 4 shows that all the copper added in water remains as dissolved species when pH 5.00, alkalinity 400 mg/l and initial copper concentration 1.5 mg/l. But the concentration of free copper decreases as the pH increases. Figure 4 and 5 shows that the concentration of free copper decreases from 58% to 27% as pH increases from 6.00 to 7.00. At pH value near 7.00, solids dominate copper speciation, except when copper concentration is very low (at or below 0.05 mg/l). As the alkalinity and/or initial copper concentration increases formation of solids also increases. At pH values near 7.0, Malachite, $Cu_2CO_3(OH)_2(s)$ is the dominant solid species. (Figure 6, 7, 8)



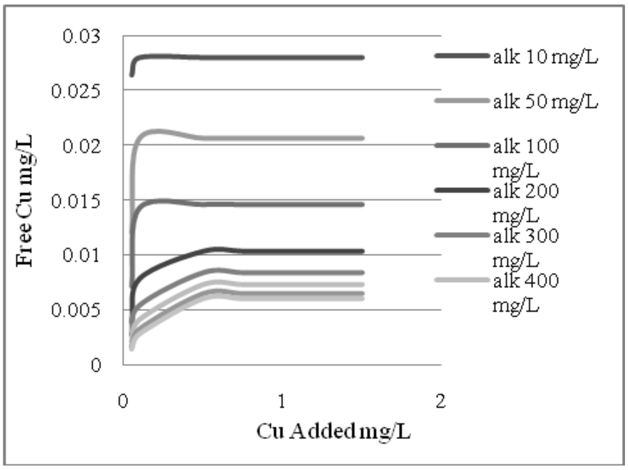
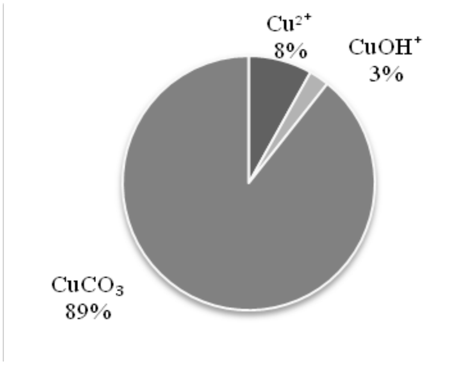


Figure 6: Relationship of initial copper with dissolved and free copper at pH 7.00 and alkalinity 10 to 600 mg/l



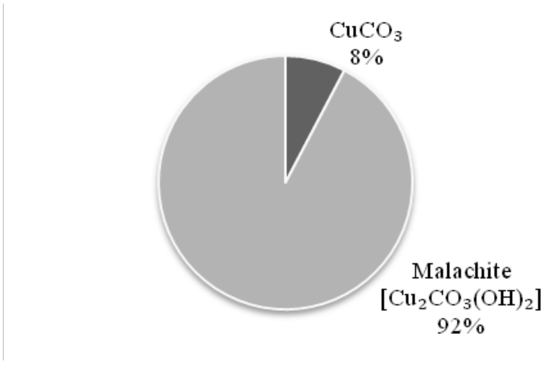
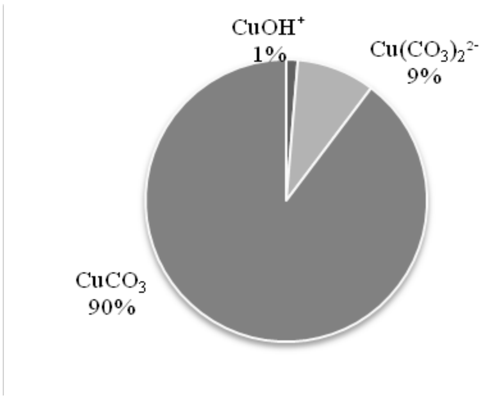


Figure 7: Different copper species at pH 7.00, alkalinity 200 mg/L and copper 0.05 mg/l

Figure 8: Different copper species at pH 7.00, alkalinity 200 mg/L and copper 1.5 mg/l

Figures 7 and 8 show the relative percentage of different copper species at pH 7.00, alkalinity 200 mg/l and initial copper concentration of 0.05 and 1.5 mg/l respectively. At lower initial copper concentration (at or below 0.05 mg/l), $CuCO_3$ (89%) is the dominant copper species. But as the initial copper concentration the concentration of free and dissolved copper decreases and the solid species (92% malachite) becomes dominant.

At higher pH value (pH≥8) the percentage of free copper (Cu^{2+}) is almost zero. All the copper present in water forms dissolved species ($CuCO_3$) and solids (Malachite and Tenorite). However, the concentration of dissolved copper remains independent of the total copper added to water; majority of copper is precipitated as Malachite near pH 8.00 and as Tenorite near pH 9.00. The concentration of dissolved copper remains constant because with the increase in alkalinity and copper concentration the amount of solids (Tenorite and Malachite) formation increases (Figure 9 to 10)



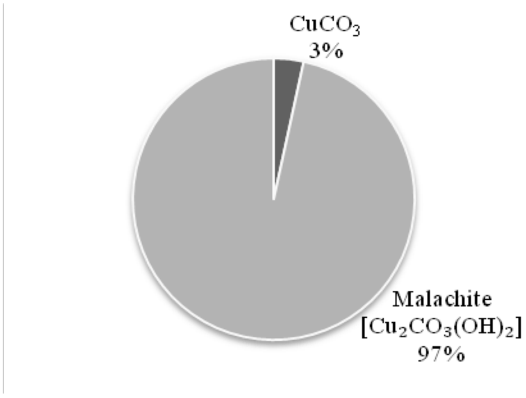


Figure 9: Different copper species at pH 8.00, alkalinity 400 mg/L and copper 0.05 mg/l

Figure 10: Different copper species at pH 8.00, alkalinity 400 mg/L and copper 1.5 mg/l

Figure 9 and 10 shows the relative percentage of different copper species at pH 8.00, alkalinity 400 mg/l and initial copper concentration of 0.05 and 1.5 mg/l respectively. At lower initial copper concentration and pH 8.00 no solid species of copper is formed. But as the initial copper concentration and pH increases solids become dominant . Malachite and Tenorite are the dominant solid species at pH 8.00 and 9.00 respectively. At higher pH and higher initial copper concentration, almost all the copper precipitates as Tenorite (99% Tenorite).

3.3 Speciation of Copper for SWTP and Potential for Algae Removal

Simulations were done to evaluate whether copper salts can be added to the raw water at the STWP for controlling algae growth. Since only dissolved Cu species are effective in controlling algae growth, the simulations were done to see dissolved copper concentration under different scenarios. The concentration of free copper (Cu^{2+}) is also assessed, because it is the most effective species in controlling algae. As described above, the major changes in concentration of free and dissolved copper occur in a narrow pH range of 5.00 to 7.00; so pH was varied from 5.00 to 7.00. Concentration of total copper was varied from 0.05 mg/L to 20.0 mg/L and the alkalinity was set to 200 mg/L, considering prevailing raw water quality at SWTP.

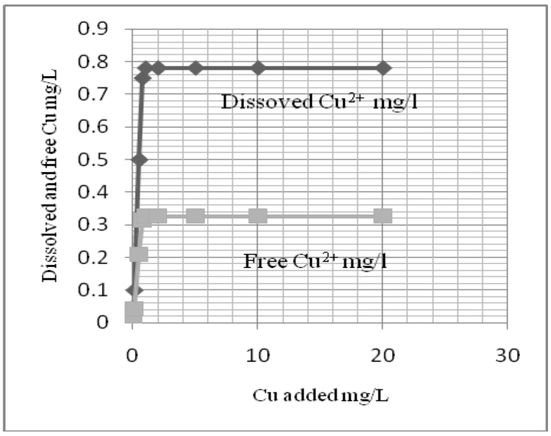


Figure 11: Relationship between initial copper with free and dissolved copper at pH 6.00

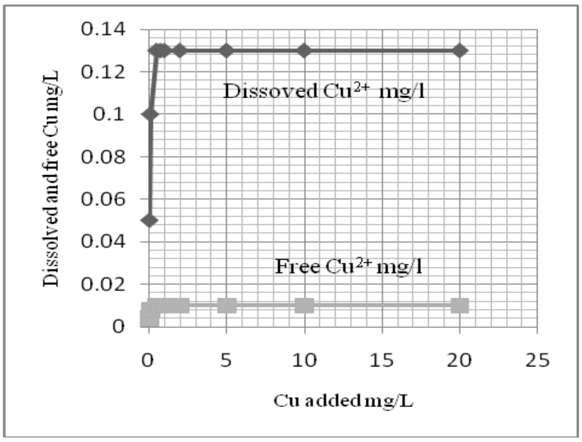


Figure 12: Relationship between initial copper with free and dissolved copper at pH 7.00

From Figure 11 and 12 it is seen that the concentration of both dissolved and free copper decreases as the pH increases. This is due to the formation of solids (Malachite) and aqueous species of copper. At a particular pH, concentration of both free and dissolved Cu initially increases with increasing total Cu added, but then remains constant irrespective of added copper concentration. Under the water quality considered, the maximum concentrations of dissolved and free copper achievable at pH 7 (close to nature pH of SWTP water) are 0.13 mg/l, and 0.01 mg/l respectively (Figure 12). These concentrations are far too low compared to those required for algae control. Hence, algae control at the SWTP is not likely to be achieved without pH control (i.e., lowering pH of water for promoting formation of dissolved/ free copper species). Lower pH would promote higher concentration of free and dissolved copper. At pH 6.0, the maximum dissolved and free Cu concentrations are 0.78 and 0.33 mg/l (Figure 11), which could be achieved with a minimum Cu dose of 1 mg/l. Even higher dissolved/ fee Cu are concentrations could be achieved if pH is further lowered. So for controlling algae using copper salt at the SWTP, a copper dose of 1.0 mg/l could be used (e.g., at the DND conveyance canal), but the pH needs to be lowered to about 6.0 with addition of acid. However, in such a case, the pH needs to be readjusted at subsequent treatment steps.

3.4 Removal of Copper by Laboratory Batch Experiment

Figures 13 and 14 show removal of Cu as a function of pH for alum doses of 50 mg/L and 25 mg/L, respectively. These figures show that Cu removal is a strong function of pH, with removal increasing rapidly with increasing pH. However, Cu removal did not change significantly with change of alum dose considered here(25 to 50 mg/l). The copper removal increases from about 10 percent at pH 4 to about 65 percent at pH 7, under the experimental conditions. So if excess Cu present in water (due to addition of Copper salt for algae control) needs to be removed by alum coagulation, the pH should be adjusted close to neutral range for effective Cu removal.

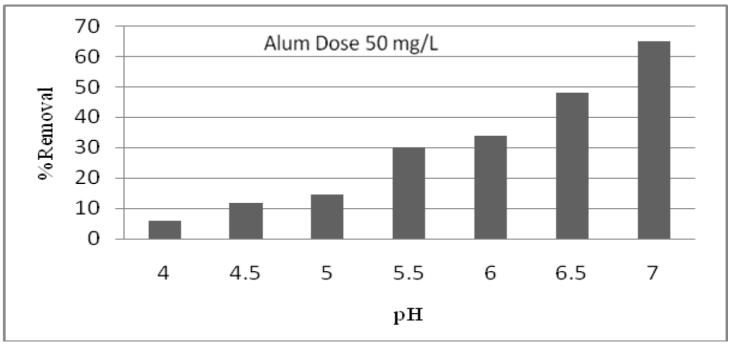


Figure 13: Removal of Cu as a function of pH by alum (50 mg/L) coagulation

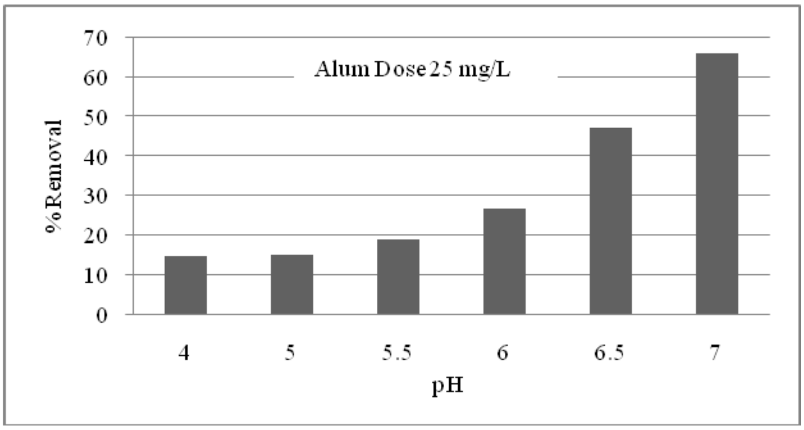


Figure 14: Removal of Cu as a function of pH by alum (25 mg/L) coagulation

4. CONCLUSIONS

The chemical speciation of Copper in natural water mainly depends on alkalinity (i.e., carbonate/ bicarbonate) and pH. At low concentration of copper (at or below 0.05 mg/l) and acidic pH values (≤pH 7), all copper exists as dissolved copper. As Copper concentration and Alkalinity increases, concentration of free copper (Cu^{2+}) begins to decrease, and formation of aqueous copper species and solids increases. At pH above 8.0, the concentration of both free and dissolved copper decreases significantly due to the formation of solids. Since dissolved copper species, particularly Cu^{2+}, are most important for controlling algae; pH ranges 5.0 to 7.0 is of particular interest with respect to algae control. Under the water quality considered(SWTP), the maximum concentrations of dissolved and free copper achievable at pH 7 (close to nature pH of SWTP water) are far too low compared to those required for algae control. Hence, algae control at the SWTP is not likely to be achieved without lowering pH of water. So for controlling the growth of algae at the SWTP, a copper dose of 1.0 mg/l could be used on a pilot basis; but the pH needs to be lowered to about 6.0 with addition of acid. However, in such a case, the pH needs to be readjusted at subsequent treatment steps. It has been found that during alum coagulation, copper removal increases as pH increases. So if excess Cu present in water (due to addition of Copper salt for algae control) needs to be removed by alum coagulation, the pH should be adjusted close to neutral range for effective Cu removal.

5. REFERENCES

Brown, D.S. and Alison, J.D. (1987). MINTEQA1, An Equilibrium Metal Specification Model: User's Manual. U.S. Environmental Protection Agency, Athens, GA. EPA/600/3-87/012.

Carbonell, G. and Tarazona, J. V. (1994). Toxicokinetics of copper in rainbow trout (Oncorhynchus mykiss). Aquatic Toxicology 29:213-221.

Schroeder, H. A., Nason, A. P., Tipton, I. H. and Balassa, J. J. (1966). Essential trace metals in man: copper. Journal of Chronic Diseases 19:1007-1034.

Schecher, W.D. and McAvoy, D.C. (1992). MINEQL+ : A Software Environment for Chemical Equilibrium Modeling, Computers, Environment and Urban System, 16:1, 65-76.

U.S. Environmental Protection Agency (USEPA) (1980). Ambient water quality criteria for copper. U.S. Environmental Protection Agency Report 440/5-80-036.162 pp.

SURFACE WATER SALINITY INTRUSION ASSESSMENT OF SOUTH-WEST REGION OF BANGLADESH: A GIS BASED APPROACH

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ABSTRACT

South-West zone of Bangladesh is located in a position very Susceptible to water resources related issues and climate change. The South-West region of Bangladesh is a low lying delta with a complex crisscross river system. From last few decades Surface water salinity intrusion in the South–West zone in our country has become one of the major concerns. The entire river system of the South-West region is Susceptible to extravagant saline water intrusion from the sea with high tide especially during dry season. This problem is exacerbated by decrease of upstream flow due to Farrakka Barrage, Sea level rise, Expansion of shrimp farms and CEP (Coastal Embankment Project) Implemented during the 1960s. These facts will lead us to acute problems like decrease in farming lands, increase in groundwater salinity and soil salinity, increased food insecurities, strong scarcity of safe drinking water and loss of biodiversity. This paper evaluates the trend of change in the surface water salinity from the year 2001 to 2008 and analyzes the reasons behind this change. Surface water salinity data source is BWDB. ArcMap 10.1 is used for this purpose combining with analytical analysis and Geostatistical analysis. This paper shows increasing trend of surface water salinity majorly due to decrease in upstream fresh water flow rather than the climate change phenomenon.

Keywords: Salinity intrusion,GIS,Geostatistical analysis, Climate change, Fresh water

1. INTRODUCTION

In all Coastal regions of Bangladesh, saline water advances and retreats on a daily, fortnightly and seasonal basis. The first two movements are in response to the diurnal and spring-neap tidal cycles, whilst the third is a response to the huge seasonal variation in fresh water flows in the estuaries. There is a zone of continuous transition between fresh water and open-sea salt water. The ecology and land use pattern of coastal regions is adapted to the normal movement of the saline front, and along most of the coast, an equilibrium regime has been established. However the regime has been affected significantly in the Southwest by the reduction in flows into the Ganges distributaries, especially the Gorai, and to a lesser extent, the Meghna estuary, in response to major freshwater withdrawals from the dry season flows of the Ganges. The effect has been particularly severe in the greater Khulna. Climate change phenomenon more aggravates the problem due to the rise of sea level. The main focus of this study is to identify the main reason and to establish a strong relationship with the driving fact.

2. METHODOLOGY

2.1 Study area

The South West zone of Bangladesh comprises of 21 districts and have very complex river network along with other water bodies like Beel, Wetlands and Khals. Mainly this zone can be divided into two parts, the coastal zone and the non-coastal zone. Among 21 districts 7 districts are in direct interaction with Bay of Bengal and can be classified as Coastal zone of South-West part of Bangladesh. Other 14 districts situated in non-coastal zone (Table 1). There is Physical, climatic and geomorphic difference between this districts.

Table 1: List of Districts, based on coastal and non-coastal zone.

Name of the Districts (Noncoastal)	Name of the Districts (coastal)
Khustia, Meherpur, Jessore, Narail, Magura, Jhenaihdah, Chuadanga, Rajbari, Faridpur, Gopalgonj, Madaripur, Shariatpur, Barisal, Jhalokati	Khulna, Satkhira, Bagerhat, Barguna, Pirojpur, Patuakhali, Bhola

In this Study area up to 2008 there are 70 Salinity Stations but due to lack of data and Latitude Longitude value only 52 stations were taken under consideration. Using Arc map 10.1 the exact location of the stations were pointed out precisely and presented in a form of map (Figure 1). This map not only shows the stations but also represent the whole study are. This map is shown below

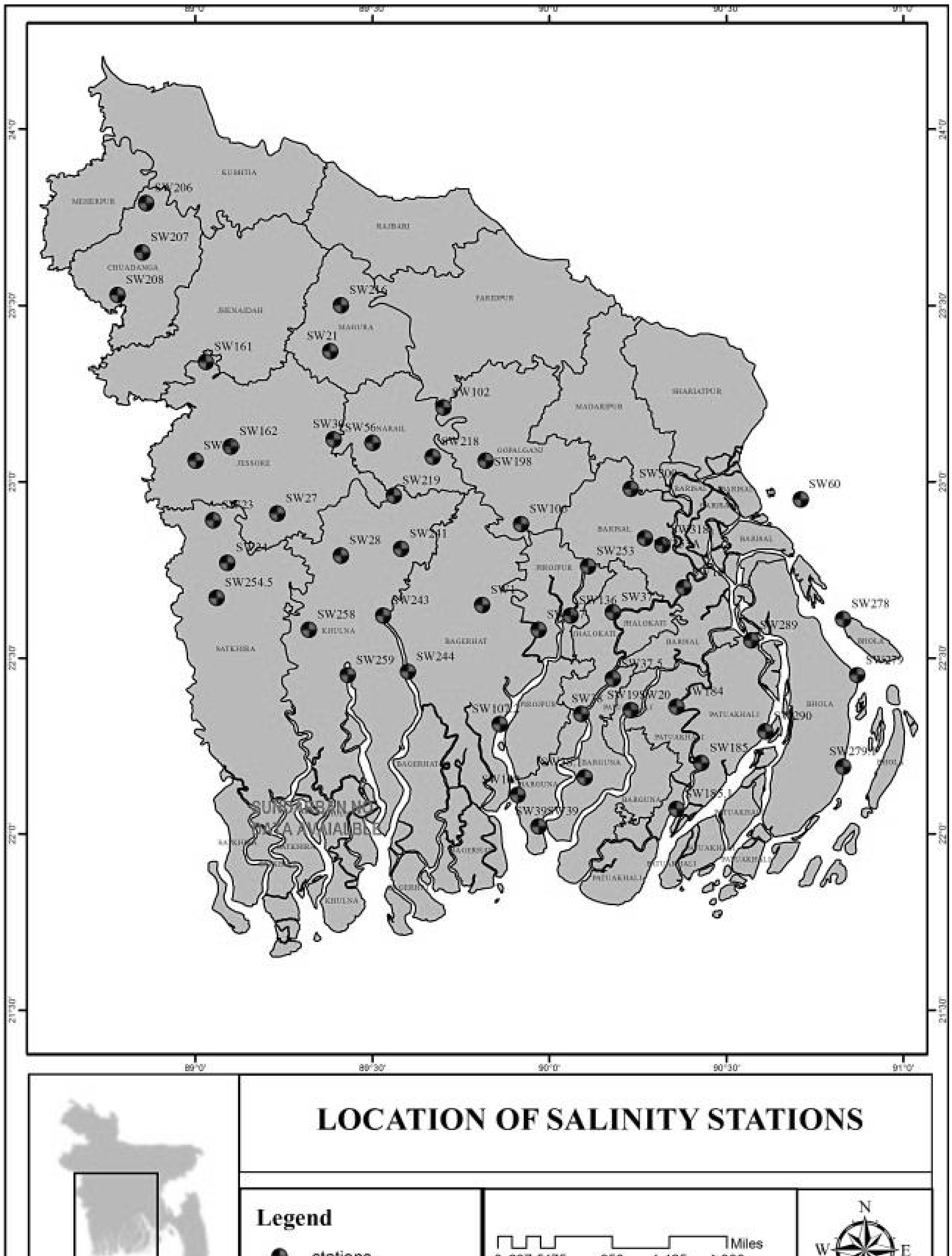

Figure 1: Location of Salinity Stations at South-West zone of Bangladesh

2.2 Data

Data used in this study was collected from BWDB. Salinity analysis of water samples collected from 52 stations (From 2001-2008) of South-Western part of Bangladesh is being performed through South-Western Measurement Division, BWDB. Out of this for 51 stations water samples are collected at high and low water levels during new moon, full moon and mid of moon. For stations at Khulna water samples are collected from Rupsa-Passur River at every high and low water levels from 1st January to 31st May and November, December and June water samples are collected at high and low water levels during new moon, full moon and mid-off moon. Measuring unit is in EC (Electric Conductivity). Here EC is converted to more commonly Water Salinity unit ds/m or mmhos/cm using relation among EC and ds/m.

EC (mmhos/cm or dS/m) x 640 = TDS (mg/l or ppm)

2.3 Interpolation Method

The interpolation methods used in this study were performed by Arc Map 10.1. Spatial Analyst and Geo-statistical Analyst is an extension to the ArcGIS Desktop that provides a powerful suite of tools for spatial data exploration and surface generation using sophisticated statistical methods. Geo-statistical Analyst provides two groups of interpolation techniques: deterministic and Geo-statistical. From the applied methods by evaluating RMS value it has been seen that Kriging method is best (less RMS value means more accurate result). But using this method the situation on the central upper part is not interpolated correctly. Interestingly it is also seen that in IDW method Sundarbon areas are projected with low value as per following the algorithm of the IDW method but this result is not also reflecting the real scenario. Spline is fitting better but if we subtract the Sundarbon area in Natural Neighbor method and interpolate, it gives a quiet convincing result. So after evaluating all available interpolation method Natural Neighbor Method is selected.

2.3.1 Saline Water Classification, Discharge and SLR

In Bangladesh Major Surface water using sector is Agriculture .So in this study saline water classification based on irrigation requirement defined by Food and Agriculture organization of the United Nations has been followed (Table 2).

Table 2: Irrigation Water Quality requirement

Hazad	TDS(ppm or mg/L)	dS/m or mmhos/cm
None	<500	<.75
Slight	500-1000	.75-1.5
Moderate	1000-2000	1.5-3.00
Severe	>2000	>3.0

Then total area is interpolated with four range of Salinity (0ds/m-.75ds/m, .75ds/m-1.5ds/m, 1.5ds/m-3ds/m, >3ds/m).This Operation has been Carried out for 8 consecutive years starting from 2001to 2008 (Figure 2). South West Estuary of Bangladesh is basically divided into Three Estuary Systems named Western Estuary System (WES), Central Estuary System (CES) and Eastern Estuary System (EES). Form the GIS based Surface Water Salinity Analysis it's clear that CES and EES is not under the coverage of Severe Surface water Salinity zone. So for the analysis of Discharge, focus is given to the source of fresh water contributor of the WES part. In the extensive Complex River network of South West Region Gorai and Arial Khan River are main contributors for the WES part. Here for Gorai Discharge Station SW99 and for Arial Khan Discharge Station SW4A's Three months Dry period Average Discharge (March-May) taken in to account staring from 2000-2001 to 2007-2008. Different organization had different SLR projection for different duration of time. Here mean of five organization Altimetry data sets (CU, NOAA, GSFC, AVISO and CSIRO) is used to show trend of change of SLR.

3. RESULTS AND DISCUSSION

3.1 Surface Water Salinity Risk Area

It has been observed that Surface water salinity is increasing with increase of time. Using logical expression Surface Water Salinity Risk map is produced for each year. This Map Divide the Total interpolated area in two zones, Severe Risk zone (Deep Color) and Non-Severe risk Zone (Light Color) (Figure 3). Area of the severe risk zones are calculated in sq. km.

Table 3: Surface Water Salinity Risk Area

Year	2001	2002	2003	2004	2005	2006	2007	2009
Risk area(sq. Km*10^3)	11.125	12.469	12.776	13.817	14.939	16.941	17.231	17.542

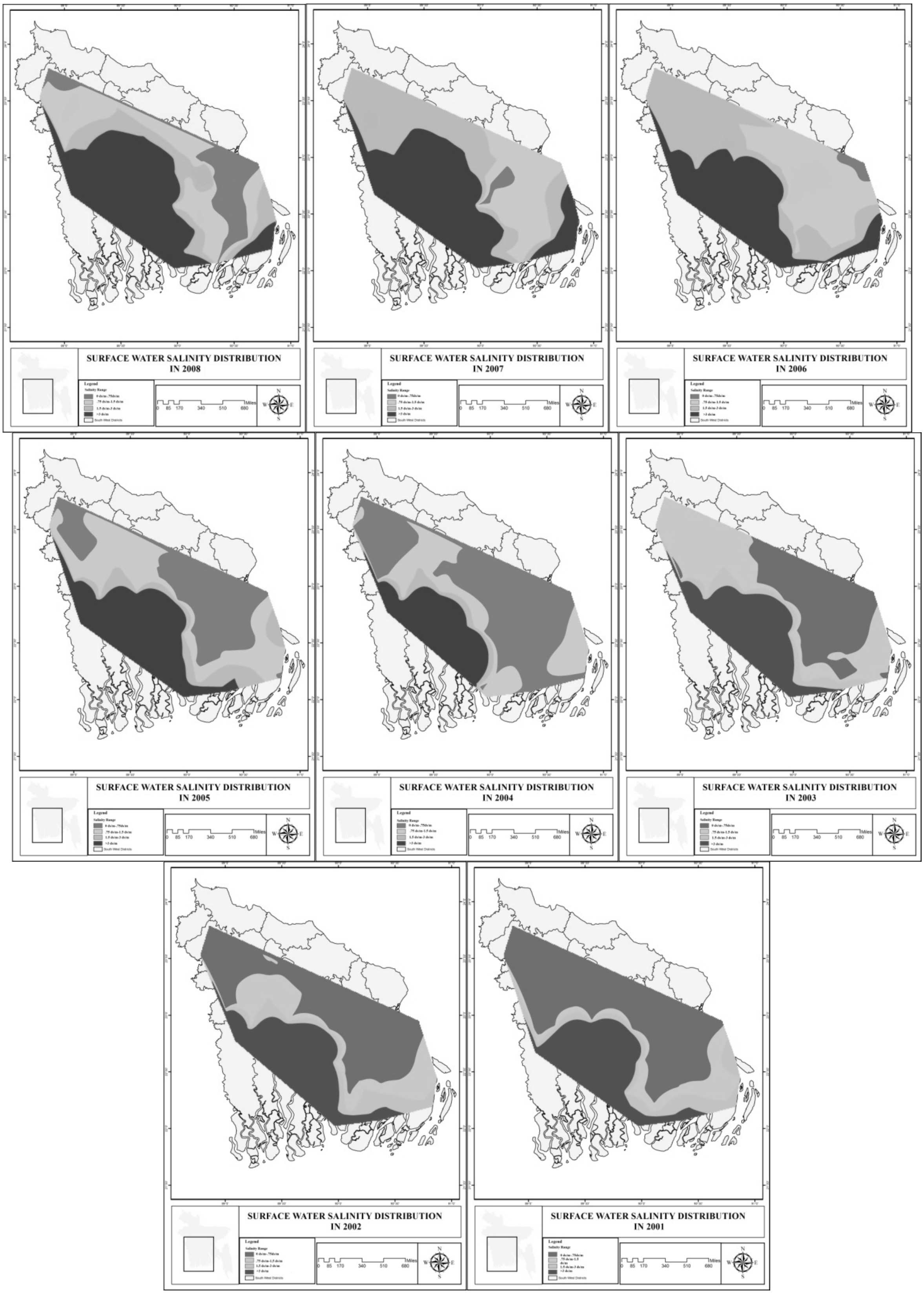

Fig 2: Surface Water Salinity Distibution in Different Year.

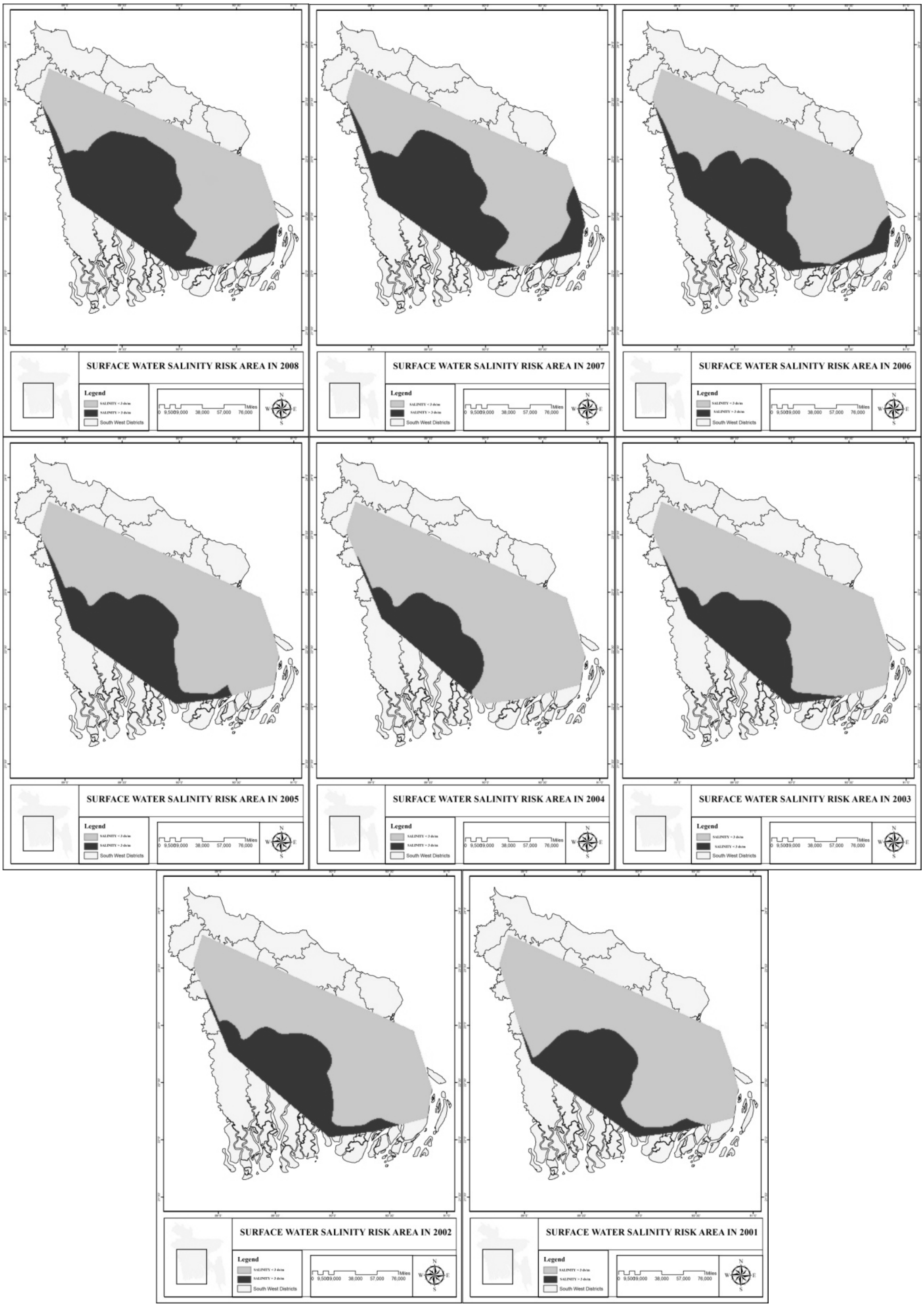

Fig 3: Surface Water Salinity Risk Maps

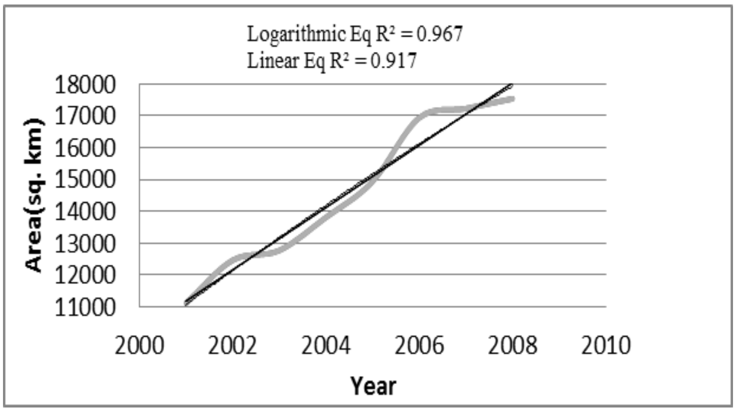

Figure 4: Surface Water Salinity Risk Area vs. Year

In Figure 4 Surface Water Salinity Risk Area Vs. Year is plotted. As Trend line is added for two types of equation (Linear and Logarithmic) results different Regression coefficient value. For Linear equation fitting R^2= .917 and for Logarithmic equation fitting R^2=.967. This R^2 values indicate that Logarithmic profile is more accurate to fit Salinity Risk area changes over the year.

3.2 Discharge

To Show an increasing trend of decreasing Discharge, Discharge value of both Stations are subtracted from a fixed value (For SW99 its 1000 and for 4A its 200). Figure 5 and 6 also clearly indicate that both station change yield good fit with Logarithmic profile rather than the linear profile.

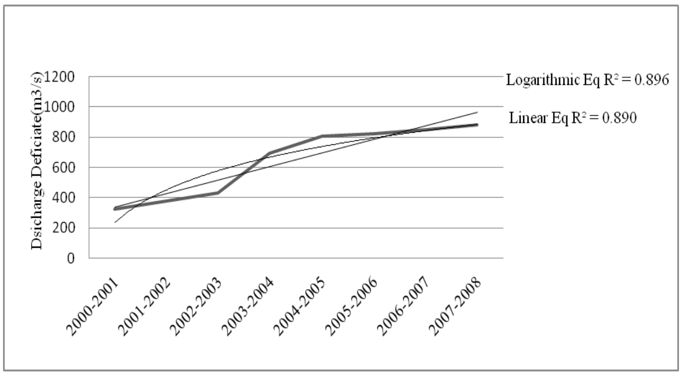

Figure 5: Average Discharge Deficit from fixed value VS. Time Graph for SW4A

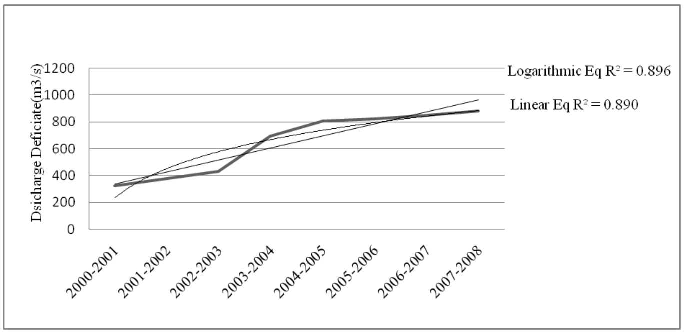

Figure 6: Average Discharge Deficit from fixed Value VS. Time Graph for SW99

3.3 Sea level rise

Different organization had different SLR projection for different duration of time. Here mean of five organization Altimetry data sets (CU, NOAA, GSFC, AVISO and CSIRO) is used to show trend of change of SLR (Figure 7).

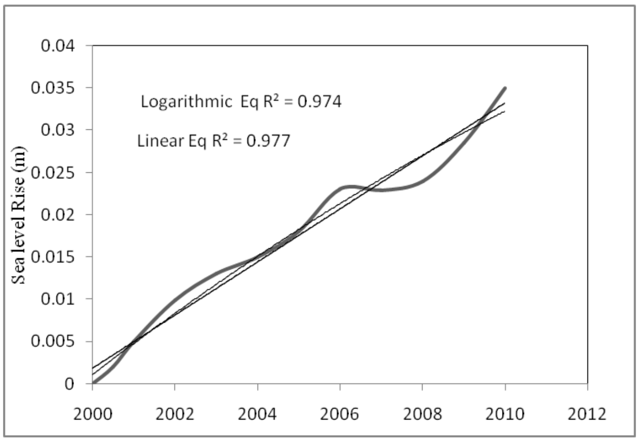

Figure 7: Sea level rise over the time

Interestingly Sea level rise follow a Liner profile (R^2=.977) better than Logarithmic profile (R^2=.974).
From the above results it is shown that Surface Water Salinity risk area and Discharge profiles are following the same Profile of increase (Logarithmic profile) whereas SLR following a Linear profile. Which strongly relate a relationship between Discharge and Surface Water Salinity and establish it as the key driving factor whereas sea level rise remain secondary driver of Surface water Salinity Intrusion in South West part of Bangladesh.

4. CONCLUSIONS

In the South West part of Bangladesh Salinity Intrusion problem is majorly due to lack of Fresh Water flow from Upstream compared to SLR. So for solution of this problem an Upstream to Downstream approach required rather than Local Approaches. It also shows that as increasing trend of Surface Water Salinity Risk area follow a logarithmic profile, first surface water salinity is increasing very sharply and then will follow a very mild slope pattern. It indicates that if the Estuary systems adjusted themselves with a constant fresh water feed in future, that mild slope of Logarithmic profile will indicate Surface water Salinity intrusion due to SLR and the amount can be quantified separately.

REFERENCES

Aurin, F.H. and Islam and A.K.M.S. (2013)"A Review of the Estimations on Sea Level Rise for the Bay of Bengal due to Climate Change" proceeding of the 4th international conference on water & Flood Management (ICWFM-2013).

De Smith, M., Longley, P. and Goodchild, M., (2011)" Geospatial Analysis- A Comprehensive guide to principles, techniques, and software tools" http: // www. Spatial analysis online. com/output/.

FAO Corporate Document Respository (n.d.)" Criteria, Standard and Considerations in the Assessment of the Suitability of Saline Water for Irrigation and crop Production" [Online]. www.fao.org/docrep/to667e08.htm

Li, J., and Heap, A.D. (2008)." A Review of Spatial Interpolation Methods for Environmental Scientists."Geoscience Australia, Record 2008/23.

Longley, P., Goodchild, M., Maguire, D., and Rhind, D. (2011). Geographic Information Systems and Science. Wiley, New York.

Rahman, M., Das, R. and Akber, M.A. (2013)"Evaluation of Dry season Water Quality of Mayur River Flowing through the Western Fringe of Khulna City" proceeding of the 4th international conference on water & Flood Management (ICWFM-2013).

Rahman. M. (2006); "Salinity Intrusion and Management Aspects in Bangladesh 2006." Journal of Environmental Hydrology, Vol.14(14), 1-8.

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CONSTRUCTION AND RENOVATION OF SEWER NETWORK AT A DEVELOPED RESIDENTIAL AREA IN DHAKA CITY: A CASE STUDY

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ABSTRACT

A trustworthy wastewater collection system is essential for human health and economic development. A case study of such work is very important for its application to other areas of the cities and towns of Bangladesh. Construction and renovation of sewer network at a selected developed residential area in Dhaka city is a very complicated job as there are no by-pass lanes in the area for proper transport. Dhaka WASA has done a renovation work during 2013 through the Sudhi Samaj Abashik Area to make a separate waste-water collection system. The aim of this study is not to find the limitations and mistakes of this development work, rather to find a better way of working procedure and how to cope with the limitations and how to make a proper use of the resources. This paper contains an extensive study on sewer renovation work and describes selected important features as well as the limitations and the effectiveness of such renovation work. The result shows that,(i) the renovation work took comparatively shortest possible time; (ii) Contract signing with two separate contractors for a same project increased the efficiency and accuracy of the work; and, (iii) A higher quality of work was achieved by full co-operation of the community.

Keywords: Construction, renovation, sewer network, residential area

1. INTRODUCTION

A trustworthy waste-water collection and treatment system is must for a country. Adequate water supply, effective waste-water collection and proper treatment are essential for human health and economic development. Installing and maintaining a well planned waste-water collection system is three times expensive than the cost of constructing fresh water supply system. Developed and industrialized countries have fairly high standard water and waste-water control system. On the other hand, sever problems in water and waste-water collection systems are apparent in developing countries (Wilderer and Schreff, 2000). The major concern of this paper is waste-water collection system. Waste-water collection and disposal system failure must cause a heavy pollution. Bangladesh is a developing country. It is the ninth most populous country and twelfth most densely populated country in the world. More over the population growth in urban area is increasing rapidly. As much the population is increasing, the problems in waste management as well as the waste-water collection system are increasing. Dhaka is the most polluted city in the world (Wikipedia). As a developing country and insufficient fund, it has become a concern to construct a high standard waste-water collection system for Bangladesh.

Dhaka WASA has given emphasis on establishing fine water supply system and waste-water collection system science 1963. Dhaka WASA is coming to the point of implementing such effective strategies to the urban area waste-water collection systems which are less costly as well as trustworthy. Waste-water collection system in Dhaka is a centralized waste-water collection system. In some areas it is combined conventional and in some areas it is separate conventional system. Clay and concrete pipes are used to convey waste-water in most of the communities in Dhaka. In order to increase the capacity of sewers and to decrease subsequent failures due to age and third party damage, Dhaka WASA is arranging renovation works among the communities in Dhaka. Sudhi Samaj Residential Area (SSRA) in Ramna Thana is one of those communities. A renovation project has done through this area during the year 2013. Previously this area was under combined conventional system. Dhaka WASA has constructed a separate conventional sewer system through this area. This paper contains the detail description of this renovation work. On the basis of collected data, a discussion on selected important features such as the effectiveness of separate conventional sewer system according to the geological and economical basis of Bangladesh, benefits of inhabitant co-operation, limitations of this renovation work are provided in this paper. This paper will be helpful for planning and projecting further renovation work in similar densely populated communities in Bangladesh. On the basis of collected data, selected important findings are concluded. To find the limitation and mistakes is not the aim of this case study, rather than the aim of this case study is to find the effectiveness and potentiality of such kind of renovation work according to the circumstance of

Bangladesh such as scarcity of financial support, weak managerial capacities and rapid increase of population as well. These circumstances are challenging the efficiency of urban drainage crisis management also (Macaitis, 1994).

2. METHODOLOGY

To discuss the renovation procedure, the community have classified in two areas basis on their roads. On the basis of primary data we made a description about the renovation procedure. Using the secondary data we discussed the limitations and effectiveness of this renovation work.

2.1 Primary Data Collection

Siddheswari Sudhi Samaj Residential area is situated at Ramna Thana of Dhaka city. The total area of this community is around 0.25 km2. From 10th April 2013 sewer renovation work had started in this community and completed at 5th May 2013. This renovation work was projected by Dhaka WASA. All primary data were collected by two procedures.

2.1.1 Field survey

We have made five times field survey to collect data. Fist time when the renovation was started, we collected the number of roads, length of the roads, average width of the roads. Second time, we collected the data about excavation, bedding and laying of pipes. Third time, data about manholes and Junctions were collected. Forth time was for backfilling. Fifth time we surveyed the situation after this renovation work.

2.1.2 Questionnaire survey

Inhabitants are the great source of primary data. Data about this community such as the situation of this community before this renovation work, population of this area, about contractors are collected by questioning. To collect data by questionnaire survey, a short list of necessary data were made and discussed about those features with the local people and landlords personally who are living in this area since the year 2000 or further past. The questionnaire survey was effective as the collected data from sever inhabitant were as far as similar.

2.1.3 Secondary Data Collection

Secondary data were collected from some important waste-water management related books, papers, journals and from internet as well. Those entire sources have cited in this paper.

3. DESCRIPTION OF SUDHI SHAMAJ RESIDENTIAL AREA

Sudhi Shamaj Residential Area (SSRA) is situated at Ramna Thana of Dhaka City. It is in the most active commercial zone of Dhaka. There are many schools, colleges, universities, shopping mall around the area. SSRA is one of the well planned residential areas in Siddheswari under Ramna Thana. Tendency of living here is very high. There are total 56 multi-storeyed buildings; most of them are above six storied. Among them there are two fourteen storied residential buildings also. There are four stationary shops, one hospital, one mosque, two laundries, two academic organization and 54 residential buildings. Around 13000 peoples are living here at present. Before 10 years during 2003 there were around 4000 peoples. The growth rate of population of this area is 23.3% per year now (Table 1). It can be estimated that after 10 years the population will be about 20000. The total area is covering 0.25 km^2 only. Therefore, the population density is very high of this area. There are four openings with highway and have two subway through SSRA. This area is restricted from high weighted vehicle (Figure 1).

Table 1: Population and growth rate

Year	Population	Population growth rate (/yr)
2000	3000	--
2003	4000	11.1%
2008	6000	10%
2013	13000	23.3%

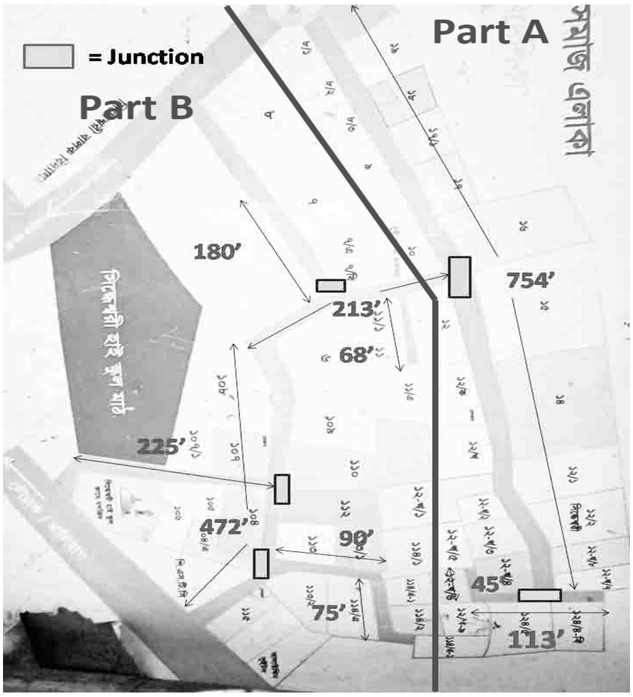

Figure 1: Map of Sudhi Samaj Abashik Area (SSRA)

SSRA is a part of Dhaka city. Therefore, the geographical condition of this area is similar to Dhaka city. Dhaka is located in central Bangladesh at 23°42′0″N 90°22′30″E, on the eastern banks of the Buriganga River. Climate of Dhaka is hot, wet and humid. The average annual temperature is 27^0C. Rainfall over the city is not uniform.

4. STUDY OF THE RENOVATION PROCESS

The whole renovation work was studied and can be explained into two different parts:

4.1 Procedures of Renovation

4.1.1 Method selection

Dhaka is under a centralized waste-water management system. Centralized wastewater collection system consists of: (1) Centralized collection system that collects waste from many waste-water producers: households, commercials areas, industrial plants and institutions and transports etc. (2) There should be a centralized waste-water treatment plant in an off-site location outside of the settlement. (3) Disposal/reuse of the treated effluent far from the centre effectively (Crites and Tchobanoglous, 1998). According to this system, the sewer of a residential area will connect to the trunk sewers and a separate convention system is preferable for SSRA.

4.1.2 Factors for strategy selection

There are some factors whether a separate conventional system will follow or a combined conventional system. Basically it depends on: the population density, quantity of water fall over a year, ratio between sewage and night soil. When there is a huge population discharges enough sewage for accelerating the night soil to obtain the minimum self velocity, and there is non-uniform rainfall through a year over the area, a separate conventional system is preferred. Moreover, to minimize the pumping and treatment cost, separate conventional system is helpful (Ahmed and Rahman, 2000). Depending upon this factors, a separate conventional sewer system have installed at SSRA.

4.1.3 Contract making

As SSRA is a small residential area covering only 0.25 km^2, only one contractor could be completed the work. However, two different contractors were assigned for this work. SSRA was divided into two parts A and B (Figure 1) for contract making. Factors and advantages of this strategy are described in the section ***7.1.1 Effectiveness of contract signing***.

4.1.4 Pipe selection

Any general material for sewer will be given considerations, but the material should be adapted to the local conditions: such as, characteristics of wastes, possibility of necessary space, soil characteristics, exceptional heavy external loadings etc (NPELF40-2005). There are no industries here. Therefore, only domestic waste will pass through this sewer system. The roads through SSRA have a width at least 10 ft. So, necessary space for a separate sewer is available. Soil condition of this area is silty clay. As heavy vehicles are restricted from this area and there is no highway or railway through this area, so the sewer will not be subjected to heavy external loadings. In this circumstance, Polyvinyl Chloride (PVC) pipes are used in this renovation work. PVC is flexible and resistance to most substances found in waste-water. Moreover, it's easy to work with PVC pipe. However, the only cause of its damaging is heavy loading and sharp edged rock.

4.2 Field Work

4.2.1 Time and working area

- Starting Date: 10th April, 2013
- Ending Date: 5th May, 2013
- Total time: 26 days
- Total length of laying sewer: 2300 feet.
- Total area of covering this sewer system: 0.25 km^2

4.2.2 Conveying of pipes

Basic materials of this construction were PVC pipe and its fittings. PVC pipes were checked carefully before accepting the shipment. Shipment is done by handcart. Pipes and their fittings were unloaded on the road. Handlings of PVC pipe were done very carefully to escape any damage and loss. Other necessary materials for the work such as: Spade, Iron-bars, Hammers, shallow pump machine etc were conveyed by pickups and stored in the tent of workers.

4.2.3 Excavation

Excavation of sewer line must be dug to grade. The depth of the line may vary with the change of surface counters. Maintenance crews used the plan view map for line and manhole location. Before beginning excavation, they also identified the nearby utility lines and to avoid any damage. They also sent a notice of this excavation to all dwellers by their buildings notice board before one week. As SSRA is a flat surface area, so the depth of trench will not vary. Traditional method for excavation was followed. They used iron-bars, hammers, spade for excavation. As the diameter of the pipe is 12 inch, they made a 24 inch wide trench, in order to make the trench ample to allow the pipe to be laid and jointed properly. Average depth of the trench was 4 ft. Trench sides were kept as nearly vertical as possible. Ledge rock, large stone, large brick chips were removed to provide a minimum clearance of 4 inches below and each side of the pipe. Excavated soils were gathered on the roadside, two handcarts were used to carry out the unnecessary soil.

Figure 2: A photograph of a trench for laying pipe line

4.2.4 Bedding

Bedding is an area requires a great care during sewer construction. Proper grading is needed to ensure a proper slope and to avoid damaging due to external loading. Sand was used for bedding as the sewer pipe was PVC. Somewhere native materials were used also due to shortage of sand. They used shallow pump machine to remove all water entered during excavation.

4.2.5 Pipe Laying and Joining

Though the bell and spigot joining is better for PVC pipe to avoid infiltration, they simply jointed the pipe by flash joining. Workers moved the pipe with hand as this were a small diameter (12 inch). It is mandatory to place a pipe in such way that the bell end faces upstream. While placing they do not follow this strictly, sometime the bell end faces upstream and some time not.

4.2.6 Slope

The rise of the pipe is 2 ft from north to south (754 ft) in part A and 1.5 ft from east to middle west (685 ft) in part B. So the slope is 1 in 385 (0.26%) for part A, 1 in 476 (0.21%) for B. This slope is very close to the recommended slope as 1 in 400 for plane topographical area (Ahmed and Rahman, 2000).

4.2.7 Service Connections

A service connection (Stub-out) is a point of contact between building sewer and waste-water collection system. Instead of making service connections they made a large number of manholes after an average interval of 40 ft. Connection were made with building sewer for existing buildings by their owner.

4.2.8 Backfill

To protect the pipe from displacement, any crush or breakage a proper compacted backfill should be done. Here native materials (Dug soil) and a small amount of sand were used for backfilling. The cover depth was about 4 ft.

4.2.9 Deflection test of sewer

No deflection and leakage test were done.

4.2.10 Manhole

400 ft distance between two manholes is preferable. Here a large number of brick masonry manholes were constructed after a minimum length 20ft to maximum 55ft. Instead of making stub-outs, they made manholes after a short distance to avoid infiltration. The manholes have average depth of 4.5 ft. Average diameter of the manholes are 40 inch and have a 20 inch wide opening for further maintenance service, made water tight by a 2 inch thick iron-cover placed at the opening of each manholes. They made a base of 1 ft larger (50 inch diameter at base) than the diameter to support the external load. There are four major cleanouts were made having a length of 4ft and width of 3 ft. 10 more small cleanouts (2 ft wide and 2 ft long) were made along the main sewer line and lateral sewer line also. Outer side of Cleanouts and manholes were plastered, inside were not plastered.

Figure 3: A photograph of a manhole

5. DETAILS ABOUT CONSTRUCTION

During the year 2000, about 3000 people were living in SSRA. Till 2008, the population growth rate was similar. After the year 2008, there is a revolutionary development done in this area. Small buildings are replaced with multi-storeyed buildings by various developers companies. As much the capacity increased, the population growth increased rapidly to 23.3% per year as shown in Table 1. The revolution have not completed yet. Still there are some high raised buildings are under construction. Therefore, there is a possibility of further population increase.

Table 2 and Table 3 show the sewer details and manholes and junction details, respectively. This sewer construction consists of three types of sewer lines. They are: Building sewer, Lateral sewer and Main sewer. Main sewer covering around 845 ft, the building sewer covering around 113 ft and the rest are lateral sewer. In total it was a 2300 ft long sewer renovation work. 116 pipes were needed to complete the project.

Table 2: Sewer Details

Item Name	Type	Location	Diameter of the pipe, d (inch)	Length of a Pipe, l (ft)	No. of Pipes, n	Total Length, L (ft)
Sewer A	Main	Part A (along western side)	12	20	38	755
Sewer B	Main	Part B (Southern-east)	12	20	5	90
Sewer C	Lateral	Part A (Southern-west)	12	20	8	158
Sewer D	Lateral	Part B (Northern-east)	12	20	9	180
Sewer E	Lateral	Part B (Through the area)	12	20	31	613
Sewer F	Lateral	Part B (Southern-east)	12	20	8	165
Sewer G	Lateral	Part B (East)	12	20	11	225
Sewer H	Building	Part B (Mid west)	6	20	2	45
Sewer I	Building	Part B (Through mid)	6	20	4	68
					Total Pipes =116	Total Length of sewer along the area = 2300 ft

Table 3: Manholes and junctions

Item Name	Length	Average width (ft)	Average depth below Ground Level (ft)	Total no.
Manhole	--	3.5	4.5	72
Cleanout/Junction	12	4	5	5

6. PREVIOUS SEWER SYSTEM

SSRA is a community established in 2002. Till 10th April, 2013 a combined sewer system was installed to collect the sewer from part A, which was connected to the Trunk sewer along the "Siddheswari Road" and "Selina Parvin Road". Pre-cast Concrete pipe were used in previous sewer system. The diameter of the pipes was18 inch. The length of the pipes was 10 ft. Thickness of the pipes was 1.5 inch. Service ability of small diameter pipes is more difficult than the larger ones. These sewers are getting old and no form of pre-emptive measures is taken. Therefore, it is inevitable that they will be subjected to a higher risk of failure in the future (Oli-Phant, 1993).

6.1 Condition before this Renovation Work

Till 10th April 2013, a combined sewer system and an open drainage were used to convey out the storm water and waste-water to trunk sewer. During the time 2003, it was sufficient because of small population. After 2003, a huge number of new multi-storeyed buildings were constructed and a rapid growth in population has seen. This huge population started to produce a large quantity of waste-water. It was not able to convey out this huge amount of wastewater for the previous sewer system and the situation became uncontrolled.

We know Bangladesh is a country of un-uniform rainfall. During the monsoon, the sewer system fails due to overloading. Roads were flooded by the water from leakages. Sometimes, night soil came out also. The area became smelly all times. In dry season, sewer system failed due to lack of minimum water because, in a conventional sewer system, wastes need a minimum quantity of water for obtaining minimum self-cleaning velocity. Moreover damages, leakages, blockages made the sewer system unusable. Water trapped here and there. Damages in sewer system also disturbed the fresh water supply system. Sewage sometimes gets diluted with fresh water. Sewage contamination in fresh water is a common scene in Dhaka. After a heavy rain, all the streets through this area go under water. It took a long time to run off the rain water because of the lacking of proper storm water drainage system. During the construction time of the buildings, the sewer system was subjected to a heavy load and somewhere the concrete pipes fail to resist the load and cracked, damaged. Due to water trapping, the roads of this area damaged gradually.

7. EFFECTIVENESS AND LIMITATIONS OF THIS RENOVATION WORK

7.1 Effectiveness of this Renovation

7.1.1 Effectiveness of contract signing with two contractors

Two different contractors were assigned to complete this renovation work. The employment of more than one contractors means that a larger pool of skilled labour resources will be available for the works. Moreover, when two contractors were assigned for similar kind of work at the same time and same place, there could be competition in reputation and effectiveness of construction methods. Such sense of “professional pride” leads the contractors to do their best effort (Tai R et. all, 2010).

7.1.2 Effectiveness of contract signing with two contractors

Bangladesh is a developing country. There is a shortage of machineries and heavy mechanical tools. Almost all development works are done by traditional methods. It implies that almost all development work is a lengthy process in our country. Moreover half done projects are common scenario. Comparatively renovation work at SSRA was done in a short time at a stress. It was completed within 26 days only. There was a total six days brake due to natural calamities (Figure 4).

Figure 4: Workers working in a traditional method

7.1.3 Advantages of separate sewer system

The sewage sewer is separated from the storm sewer. So, during monsoon, storm water cannot get diluted with sewage. It will reduce the overflow from manholes also. As, there is no uniform rainfall through the year, so separate sewage sewer is desirable. After this renovation, the smelly situation of this area is reduced. Moreover, the sewer system was installed before the actual development of this area. Then the population increasing rate was considered 11.1% per year. After 2008, population increased at a rapid rate 10% per year (Table 1). This rapid growth in population was resulting failure to the previous sewer system. So it became mandatory to extend the capacity of the sewer system.

7.1.4 Re-use of old sewer line

To extending the capacity, instead of changing the whole system, a new separate sewer system is installed for sewage only. The old system is treating as the storm sewer now. It will reduce the treatment coast. Now the sewage will be treated only. The storm water can be exposed without any treatment.

7.1.5 Cooperation from inhabitants

Most of the residents living in this area are well educated. A good co-operation between workers and the residents have seen. Due to the awareness of residents, the workers couldn't do any fraud. Residents have to sacrifice some of their rights during this renovation work. As an example, they have to manage a new parking lot for their vehicles till the road was re-constructed after this renovation. Educated and aware people are an asset of a country. They are able to sacrifice in order to development work of their country. In our country, most of the construction or renovation works have to face an interruption from the inhabitants due to lack of their knowledge and awareness.

7.2 Limitations

7.2.1 Insufficient tools for working purpose

Shortage of tools for this work had observed. Workers hadn't proper excavation tools. Both assigned contractors had one sallow pump machine to remove the water from trench. There were only two handcarts to carry out the excavated soils.

7.2.2 Shortage of space

SSRA is a densely populated area. There was no open field such as playground in SSRA for keeping construction materials. There was not enough space in road side. So they had to use the road to keep construction materials. Even the excavated soils were gathered on the road beside the trench also. (Figure: 5).

Figure 5: Excavated soils are gathered on the road

7.2.3 Lack of protective measures

There was no railing stuffed or any other protective measure beside the trench. So, there was a possibility of occur any accident. (Figure: 2)

7.2.4 Unexpected interruptions due to natural calamities

Due to rainfall the work had interrupted. We know most of the natural calamities happen during the summer in our country. April is the summer time here. So, this renovation work had to face this type of sudden natural calamities also.

7.2.5 Making wrong slopes while placing pipes

Due to lack of proper supervision, workers failed to lay these pipes according to the proper slope. Somewhere, they made wrong slope also. There was a plan of laying pipes minimum 5 ft below from the formation level (FL) of the roads. However, they placed the pipes average 4 ft depth. To gain the minimum depth, the FL of the road will by higher. It will result a minimization of the difference between the plinth level (PL) and the formation level (FL). In some cases, the FL becomes higher than the PL.

7.2.6 Lacking of water tightness of manholes

Both inside and outside plastering is needed for acquiring the water tightness of manholes. Manholes were not plastered properly. Therefore, proper water tightness of those manholes was not ensured. This will result infiltration. (Figure: 3)

7.2.7 Backfilling quality

Sand is a good material for backfill. After laying the pipes, trenches were filled with native materials with a small amount of sand. Moreover those materials contained sharp edge large rock and brick-chips. These rock and brick-chips will cause damage to those PVC pipes. After all, it was done to reduce the cost.

7.2.8 Lack of compactness

Workers simply filled the trench but compaction was not done properly. Due to lack of well supervision, proper compactness wasn't done.

7.2.9 Lack of risk minimizing

No risk management plan was applied for both workers and properties of this area. Workers were not provided gloves, rubber boots, helmets etc. During work they had to go a direct contact with night soils. During this renovation work, damages caused to the fresh water pipes as well as the electric pole also.

7.2.10 Lack of record keeping

No record is kept and provided about this sewer renovation. So it is become hard to indentify the location of sewer below the ground. No one knows that where the pipe is or what is the size of the pipe. So there is a chance of interfere with the sewer while further construction; and often the absence of linkage between fixed asset financial data base and maintenance management records (Grigg 1994)

8. BANGLADESH AND CONVENTIONAL SEWER SYSTEM

First sewage system in Dhaka city was established in 1923. At that time Dhaka was a small city. In 1963, Dhaka WASA was established for water supply and sewage disposal. Till then, Dhaka WASA is extending its network over the city. The only sewage treatment plant in Bangladesh is in 'Pagla' started its journey from 1992. Its capacity is 120,000 m^3/day and disposes waste water into Buriganga River. Dhaka city is under a centralized wastewater collection system. Among them some communities are experiencing separate conventional system, most are combined conventional system and some where there is an open drainage system as well. Clay and concrete pipes were used in most of the sewers.

9. CONCLUSIONS

From the study the following conclusions can be drawn:

According to the study with specific emphasise on effectiveness and the geological conditions of Bangladesh, this type of renovation works on sewerage network are essential for all cities and towns of this country.

Dhaka is experiencing a non-uniform rainfall. To avoid overflow from sewage sewer separate conventional sewer system is helpful. Moreover, when storm water is not contaminated by sewage, the storm water can be disposed directly to receiving water body without any treatment. As a result, the treatment cost can be reduced. Capacity of sewer system should be maximized to avoid future failure. Separate conventional wastewater collection system should be implemented through all community of Dhaka city.

Like most other countries there is a lacking of record keeping about the sewers beneath the ground. There is no proper record about the location of pipe lines which may be interfere further developments along the line. Therefore, it is necessary to have a location map after completion of the work.

The total renovation work of sewer network with length 2300 ft took duration of 26 days and the whole renovation work including construction of separate sewer and road pavement took 75 days. It took relatively the shortest possible time.
Contract signing with two separate contractors for a same project increase the efficiency and accuracy of the work. To avoid any interruption by unexpected rainfall, this type of renovation works can be done at spring. A proper risk minimizing plan should be implemented for both the properties of the area and the workers. Necessary supervision is needed so that the contractors or the workers cannot do any fraud.

Another important finding of this study is that, the community people were fully cooperative with the project work from its beginning to end, which make the project successful in a shortest possible time. Due to their awareness regarding the project, comparatively a higher quality of work was achieved.

There are limitations of necessary tools. Workers have to work in a traditional old aged method till now in Bangladesh. In such negative condition, Dhaka WASA is trying to lead this kind of renovation work with a motto of making an effective waste-water collection system through the Dhaka city.

REFERENCES

Ahmed and Rahman (2000). Water supply and sanitation rural and low income urban community. ITN-BUET, Dhaka, Bangladesh.

Grigg N.S. (1994). Maintanence management system for urban drainage. In W.A. Macaitis (Ed), Urban drainage rehabilation programmes and techniques (pp 2-10). New York: ASCE.

Macaitis W. A. (1994). Urban drainage rehabilitation programs and techniques. Task committee on urban water infrastructure failure diagnostics and remedial measures and task committee on urban drainage rehabilitation programs and techniques. New York: ASCE

NPELF40 (2005**).** Chapter 20, Design of conventional gravity sewers. [Link: http://www.deq.state.ms.us/Mdeq.nsf/pdf/SRF_NPELF40-20/$File/NPELF40-20.pdf?OpenElement] Accessible Date: 6th November, 2013.

Oliphant R. J. (1993) Identifying the long term research requirements of the water industry in the pipe technology area. UK: Foundation for water research FR 0406/

Tai R., Chan A., Seit R., (2010). planning of deep sewage tunnels in Hong Kong, Drainage service Department, Government of Hong Kong SAR, China [Link: http://www.dsd.gov.hk/EN/Files/publications_publicity/other_publications/abstracts_papers/Paper%20on%20Planning%20of%20Deep%20Sewage%20Tunnels.pdf]

Wikipedia. [Link: http://en.wikipedia.org/wiki/Waste_management_in_Bangladesh] Accessible date - 2th November- 2013.

Wilderer, P. A., Schreff, D., (2000). Decentralized and Centralized Wastewater Management: a Challenge for Technology Developers, Water Science and Technology, 41 (1), 1-8.

AMMONIA REMOVAL FROM WATER USING SYNTHETIC ZEOLITE

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ABSTRACT

Presence of high concentration of ammonia is common for many rivers in Bangladesh that receive untreated domestic sewage. Effluent of many wastewater treatment plants often contain high ammonia. High ammonia in raw water drawn from the Sitalakhya river during dry season is a major concern at the Saidabad Water Treatment Plant (SWTP); it interferes with the treatment processes at the SWTP. In this study, ammonia removal efficiency of synthetic Zeolite (Cation exchanger) from water has been assessed both in batch and column experiments. Zeolite has been found to be very efficient in removing ammonia from water. Ammnia uptake by Zeolite has been found to be a strong functin of contact time; results of batch experiments show that at low contact time (2~5 minutes) ammonia uptake capacity of Zeolite was almost half of that at higher contact time (10~30 minutes). In column experiment designed with a contact time of about 3.3 minutes, ammonia uptake capacity of Zeolite bed was estimated at 2400 mg per kg of Zeolite. Similar capacity was also estimated from batch experiments for similar contact time. After regeneration with caustic NaCl solution, ammonia removal capacity of Zeolite bed was estimated at 2573 mg of ammonia per kg of Zeolite. Thus, it appears that Zeolite bed could be repeatedly used for ammonia removal without deterioration of its capacity. Apart from ammonia, Zeolite bed has been found to have significant capacity to remove major cations, including K^+, Ca^{2+} and Mg^{2+} from water; thus Zeolite could also be effective in removing hardness from water. However, high concentration of cations in water is likely to reduce ammonia removal capacity of Zeolite bed.

Keywords: Ammonia removal, Zeolite, Cation exchanger, SWTP, Wastewater treatment

1. INTRODUCTION

Domestic wastewater and effluent from many industries contain high concentrations of ammonia. If the wastewater/effluent is not treated properly before disposal, it can result in high concentrations of ammonia in the receiving water body, which can be toxic to aquatic species and may promote eutrophication. Also, ammonia present in the drinking water is harmful for human health.

The Saidabad Water Treatment Plant (SWTP), with a combined capacity of 450 MLD (Phase I and Phase II, each with a capacity of 250 MLD), is the largest surface water treatment plant in the country. . The SWTP draws raw water from Sitalakhya River through an intake located at Sarulia and the water is conveyed to the SWTP through the DND conveyance canal. However, the water quality of Sitalakhya River and the DND canal deteriorates progressively from October to March as dry season progresses, and characterized by high concentrations of BOD, COD, TOC, Ammonia, Phosphate and algae, and low concentrations of DO. The high concentrations of algae and ammonia in the raw water are major water quality concerns at the SWTP during the dry season. Pre-chlorination used at the SWTP (for pathogen removal) also promotes removal of algae by scorching its air sac. However, due to the presence of excess ammonia in raw water during the dry season, virtually no free chlorine exists in water after pre-chlorination (at the chlorine dose used at the SWTP) due to formation of chloramines. As a result, algae removal efficiency is significantly reduced at the SWTP. Because of this, an ammonia removal unit had to be added to the SWTP during construction of Phase II in 2012. But ever-increasing ammonia concentration in raw water (reaching as high as 20 mg/l during the dry season) is still a major water quality concern at the SWTP.

According to Bangladesh Standard (GoB, 1997) the allowable ammonia in drinking water is 0.5 mg/l. Apart from natural water, higher level of ammonia in effluent from sewage treatment plant (employing secondary treatment) is also a major problem. There are various techniques for reducing ammonia concentrations in contaminated water. The most common procedures are aeration/air stripping, biological filtration by nitrification, water plants/algae ponds, and ion exchange. Among these, ion exchange by Zeolite offers a number of advantages including ability to handle shock loadings and the ability to operate over a wide range of temperatures (Jorgensen and Weatherly, 2002). Also Zeolite requires lower contact time over biological treatment of ammonia. This study focuses on removal of ammonia from water using Zeolite. Batch experiments were done to analyse the approximate capacity of Zeolite to remove ammonia. Two extensive column experiments were carried out to assess the capacity of Zeolite under continuous flow. Results from the laboratory experiments suggest that Zeolite can be used to remove ammonia to a satisfactory level, and the ammonium saturated Zeolite bed can be regenerated by using caustic Sodium Chloride (NaCl) solution for repeated use.

2. METHODOLOGY

Ammonia removal by Synthetic Zeolite was assessed in both batch experiments and in columns. The objective of the batch experiments was to make a quick assessment of the ability of the synthetic Zeolite to remove ammonia from water. Also, the effect of contact time and basic design parameters for building the columns were estimated from the results of batch experiments. Subsequently, column experiments were carried out to make detailed assessment of ammonia removal by Zeolite under continuous flow conditions.

2.1 Batch Experiment

Batch experiments were carried out in five 500 ml beaker each containing 250 ml water sample. The water samples were prepared by spiking groundwater with ammonia stock solution to attain desired ammonia concentration. Then 5 gm of Zeolite (with specific gravity of 2.1) was added in each beaker and stirred with a mechanical stirrer for 2 to 30 (2, 5, 10, 20, 30 minutes) minutes. Then water samples were collected from the top of the beakers with a pipette, and tested for ammonia. Tests were conducted under different contact time to assess effect of contact time on Zeolite capacity to remove ammonia from water.

2.2 Column Experiment

For detailed assessment of Ammonia removal by Zeolite under continuous flow conditions, column experiment was carried out. A transparent acrylic glass filter column (Fig. 1) with following specification was constructed for this purpose:

Acrylic Glass Filter Column:

Interior diameter of column	=	6 cm	(2.4 inch)
Exterior diameter of column	=	6.4 cm	(2.5 inch)
Bed section area	=	29.2 cm^2	(4.52 $inch^2$)
Thickness of plastic sheet	=	0.2 cm	(0.05 inch)
Height of column	=	99 cm	(39 inch)

Zeolite bed specification:

Zeolite bed height	=	10 cm	(3.94 inch)
Zeolite bed volume	=	292.25cm^3	(17.82 $inch^3$)

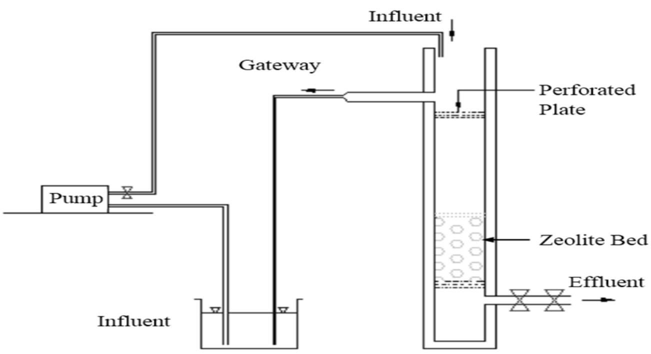

Figure 17: Schematic Diagram of Zeolite Column

Commercial grade Zeolite available at the local market was procured for the column study. Bottom of Zeolite bed was supported by a porous filter. Height of the zeolite bed was selected keeping in mind the effect of contact time; the estimated contact time was 3.3 minutes. A static water head 31.5 inch on top of the Zeolite bed was always maintained by installing a gateway outlet. Influent water for the filter column was prepared by spiking ground water collected from a deep tubewell (at BUET) with ammonia. Both influent and effluent water quality were monitored periodically. pH, EC, total Ammonia (as NH_4^+), Sodium ion (Na^+), Potassium ion (K^+), Calcium ion (Ca^{2+}), Magnesium ion (Mg^{2+}), Chloride (Cl^-), Alkalinity (as $CaCO_3$) and Hardness (as $CaCO_3$) of influent and effluent samples were measured.

For the column experiment, it was decided to close the outlet after the end of each day's operation, in order to keep the bed wet and to create a condition similar to continuous flow. It was found that if the outlet valve remains open after each day's operation the bed became dry, which results in the shock loading of ammonia in the effluent at the beginning of next day.

2.3 Regeneration of Zeolite Bed

With continued use, the capacity of Zeolite to remove ammonia becomes exhausted due to the exchange of ammonium ion (along with other cations) that replace sodium ions on Zeolite. In order to reuse the exhausted Zeolite bed, it was regenerated by sodium ion (Cooney et al., 1999). For this purpose, a solution of caustic NaCl (0.6 M NaCl solution adjusted to pH 10 adding NaOH) was passed down through the column at a rate of about 70 ml/min for 120 minutes. This process replaces the ammonium ion and other cations adsorbed by Zeolite with sodium ion (Na^+). Due to excess Chloride residue in the Zeolite bed, sufficient volume of distilled water (about 6~8 L or 20~25 bed volume) was passed through the Zeolite bed after completion of regeneration.

3. RESULTS AND DISCUSSION

3.1 Batch Experiment

Ammonia removal depends on three primary factors, e.g. Zeolite quantity, presence of sodium ions and contact time. Naturally with increased quantity of Zeolite greater amount of ammonia is removed. As NH_4^+ replaces Na^+ in the removal process, so with the increase of sodium ions in the raw water, ammonia removal efficiency of Zeolite decreases (as exchange sites available for ammonium ion reduces). These two factors remaining constant or favourable, the dictating factor influencing the removal efficiency is the contact time.

Figure 2 shows ammonia uptake by Zeolite as a function of contact time in batch experiment. It shows that increased contact time increases ammonia removal because longer exposure of Zeolite to ammonia bearing water, which increases the probability of higher exchange. Ammonia uptake by Zeolite varied from about 2500 mg Ammonia per kg Zeolite for a contact time of 2 minutes to about 5700 mg Ammonia per kg Zeolite for a contact time of 30 minutes.

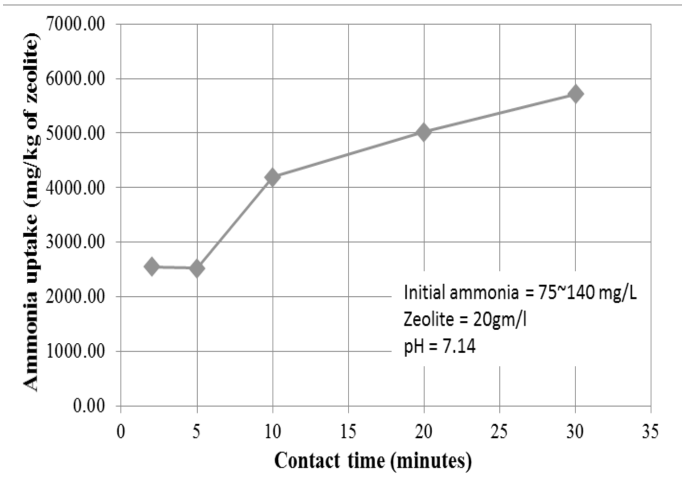

Figure 2: Effect of Contact Time on Uptake Rate of Zeolite

3.2 Column Experiment

Ground water from a BUET deep tubewell was spiked with ammonia stock solution (prepared with NH_4NO_3 salt) and was passed through the prepared Zeolite column. Table 1 shows the characteristics of groundwater used in the experiments (before spiking with Ammonia).

Table 1: Characteristics of Groundwater used in the Experiment (before spiking with Ammonia)

Parameters	Quantity
Ammonia (as NH3-N)	0.38 mg/L
pH	7.63
EC	485 µS/cm
Na^+	32.4 ppm
K^+	2.9 ppm
Ca^{2+}	53.2 ppm
Mg^{2+}	24.2 ppm
Cl^-	70 mg/L
Alkalinity (as $CaCO_3$)	215 mg/L
Hardness (as $CaCO_3$)	246 mg/100ml
TDS	350

Figure 3 and Figure 4 show ammonia concentration in the effluent as a function of bed volume of water passed through the filter column (i.e. breakthrough curve) before and after regeneration of Zeolite bed, respectively. In both cases, the concentration of ammonia in the influent was kept to around 30 mg/L. Initially there was no ammonia in the effluent; detectable Ammonia was found after passage of about 6.3 bed volumes (1 bed volume = 292.25 ml of influent) for the first run, and after about 57.6 bed volumes in the second run (after regeneration). This variation might be due to regeneration which supplied large amount of sodium ion to the Zeolite replacing all cations adsorbed in the first run. However, in both cases, significant Ammonia was not detected in the effluent before 100 bed volumes of water.

As shown in Fig. 3 and Fig. 4, with the increase in bed volume the ammonia level in the effluent also increased. The variation of effluent Ammonia concentration appears to be more smooth and uniform for the second run (after regeneration), compared to the first run. This could be due to the fact that the first run was carried out intermittently, with breaks between runs; while such breaks were less during the second run. However, the saturation limit (i.e. breakthrough point) for both runs was more or less the same.

The ammonia uptake capacity of the Zeolite bed in the column was calculated, and found to be about 2400 mg ammonia/kg of Zeolite for the first run and about 2573 mg ammonia/kg of Zeolite for the second run. For similar contact time, the capacity derived from batch experiments (Figure 2) was almost the same. This indicates that batch experiment is useful for estimation of capacity of Zeolite. It should be noted that the influent water used in this experiment contained relatively high concentrations of a number of cations (including K^+, Ca^{2+}, Mg^{2+}) that are also exchanged by the Zeolite media; this appears to have reduced the Ammonia removal capacity of the Zeolite bed.

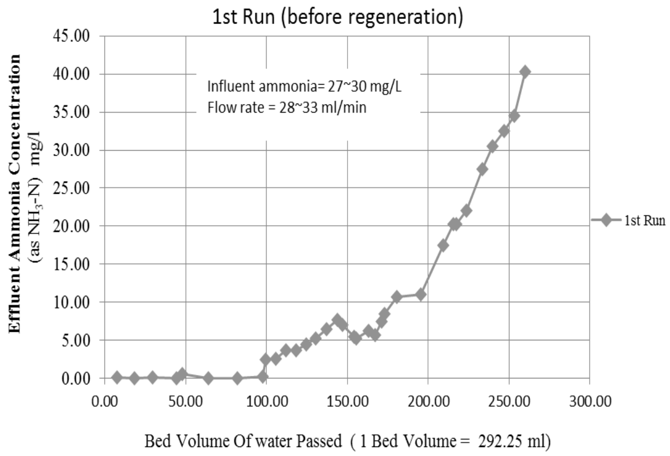

Figure 3: Removal of Ammonia (before Zeolite bed Regeneration)

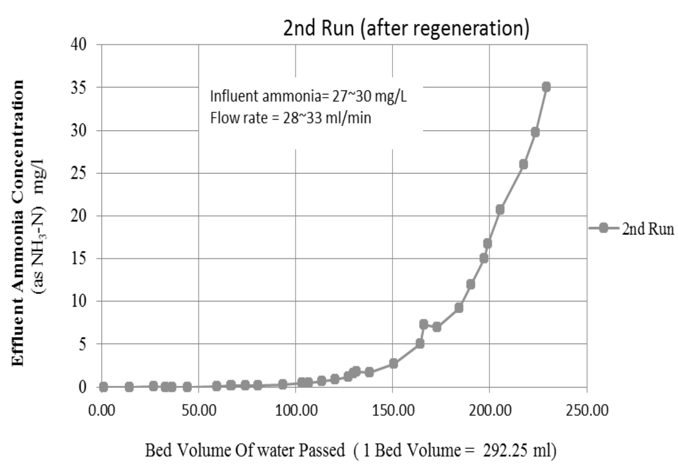

Figure 4: Removal of Ammonia (After Zeolite bed Regeneration)

Apart from ammonia, Zeolite has the ability to remove other cations (e.g. K^+, Ca^{2+}, Mg^{2+}) from water; thus the power to soften water. Figures 5, 6 and 7 show removal of K+, Ca^{2+}, and Mg^{2+}, respectively by Zeolite in the column experiments. The uptake capacity of Zeolite was estimated to be 430 and 405 mg Potassium/kg of Zeolite, 7630 and 7321 mg Calcium/kg of Zeolite, 2452 and 2353 mg Magnesium/kg of Zeolite before and after regeneration, respectively. It was also noticed that uptake of Ca^{2+} and Mg^{2+} ions by Zeolite bed was very efficient, and their concentrations remained very low even after breakthrough for ammonia was achieved. Thus, Zeolite can also be used for removal of Hardness from water; Figure 8 shows removal of Hardness by the Zeolite bed. Zeolite bed does not have much effect on concentrations of major anions in water (e.g. Cl^-, HCO_3^-).

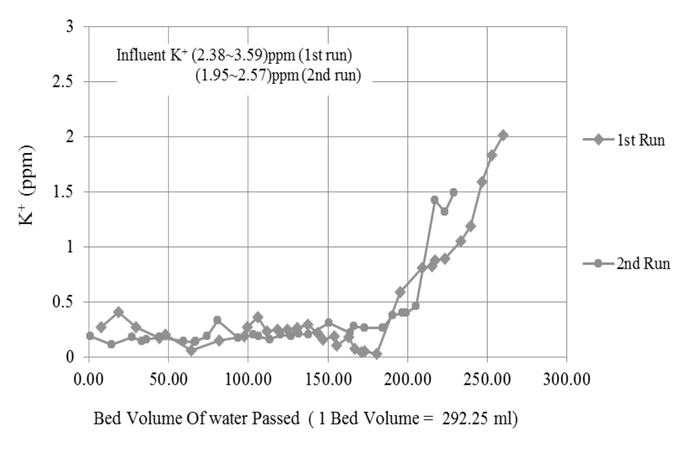

Figure 5: Removal of Potassium Ion (K^+) by Zeolite bed

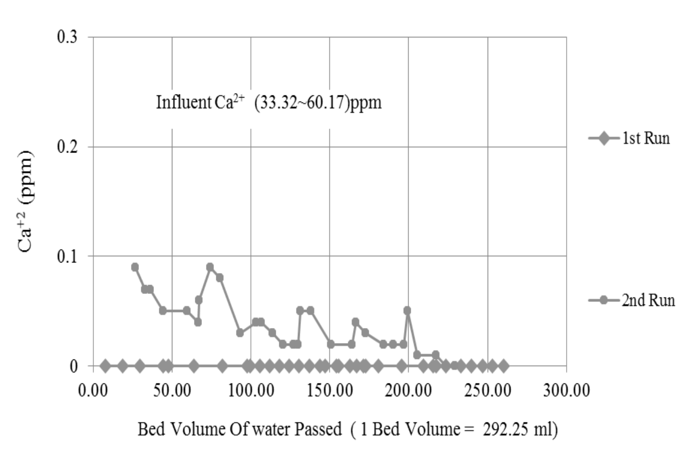

Figure 6: Removal of Calcium Ion (Ca^{2+}) by Zeolite bed

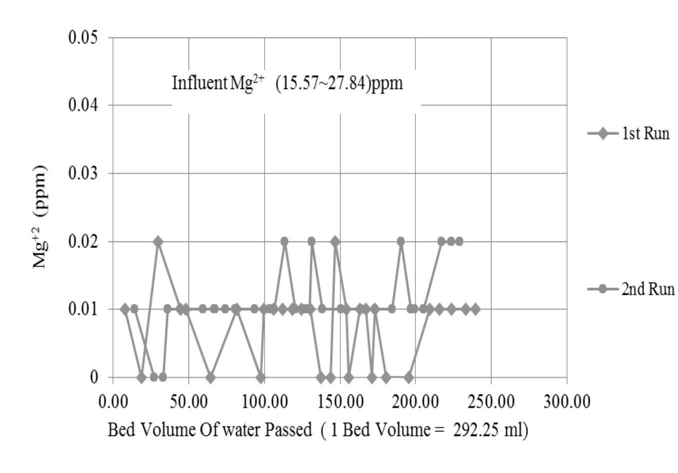

Figure 7: Removal of Magnesium Ion (Mg^{2+}) by Zeolite bed

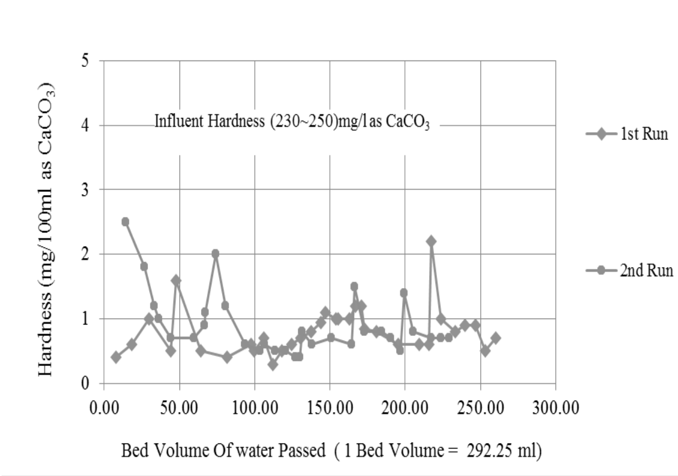

Figure 8: Removal of Hardness (as CaCO3) by Zeolite

CONCLUSIONS

The batch and column experiments revealed a number of important conclusions regarding the ability of Zeolite bed in removing Ammonia and a number of cations from water:

- Synthetic Zeolite is very efficient in removing ammonia from water. Contact time has significant influence on the removal efficiency.
- In column experiment, ammonia uptake capacity of Zeolite was found to be almost the same before and after regeneration. This suggests that the Zeolite bed could be repeatedly used for removal of ammonia without deterioration of its capacity.
- Ammonia uptake capacity of Zeolite estimated from both batch and column experiments under similar contact time were comparable, which indicates that batch experiments are useful for quick and preliminary estimation of uptake capacity.
- Apart from ammonia, Zeolite bed has been found to have significant capacity to remove the major cations, e.g. K^+, Ca^{2+}, and Mg^{2+} from water. Thus Zeolite appears to be also effective in removing hardness from water. High concentration of cations in water is likely to reduce ammonia removal capacity of Zeolite bed.
- Zeolite bed does not have much effect on concentrations of major anions in water (e.g. Cl^-, HCO_3^-). The pH slightly increases due to uptake of NH_4^+ ions from water.

REFERENCES

Cooney, E. L., Booker, N. A., Shallcross D. C. and Stevens, G. W. (1999). “Ammonia Removal from Wastewater Using Natural Australian Zeolite. II. Pilot-Scale Study Using Continuous Packed Column Process.” Separation Science and Technology, 34(14), pp.2741-2760.

GoB (1997). Environment Conservation Rules (ECR), 1997 (Information of special incident, Schedule 3), Government of Bangladesh.

IWM-DWASA (2006). Resource Assessment and Monitoring of Water Supply Sources for Dhaka City, Institute of Water Modeling and Dhaka Water Supply and Sewerage Authority, Final Report.

Jorgensen, T.C. and Weatherly, L. R. (2002). “Ammonia Removal from Wastewater by Ion Exchange in The Presence of Organic Contaminants”. Water Research, 37(2003) pp.1723-1728.

PERFORMANCE EVALUATION OF CENTRAL EFFLUENT TREATMENT PLANT IN DEPZ, SAVAR, BANGLADESH

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ABSTRACT

Wastewater management is a concept involving several operations, which depend on the scale and nature of industry or institution and end use of wastewater. Central effluent treatment plant (CETP) is an approach of treating effluents by means of a collective effort mainly for a cluster of small scale industrial units. A biometric CETP was launched in Dhaka Export Processing Zone (DEPZ) and started operation for wastewater treatment in February 2012. It was the first of its kind in Bangladesh to utilize a Bio-Electric treatment profile on a co-mingled industrial wastewater stream with an average wastewater inflow of 1750 m^3/hour. The treatment system offers tremendous advantages. Operating costs were low compared to chemically enhanced treatment and provided substantial difference (as much as 85% less) in sludge generation. It also had positive removal impacts on other contaminants that might appear as a part of total (co-mingled) wastewaters entering CETP from DEPZ factories. In this case, the use of chemicals was totally avoided and hence not required any neutralization as well as minimization of secondary pollutions by added chemicals. The present study was undertaken to evaluate the performance of the biometric CETP especially with regards to effluent water quality. Wastewater samples were collected from Primary and Secondary stages of treatment units and the major water quality parameters such as pH, Biochemical Oxygen Demand (BOD), Chemical Oxygen Demand (COD), Total Suspended Solids (TSS) and Total Dissolved Solids (TDS) were analyzed following the standard methods for the examination of water and wastewater. The Primary-stage treatment operation was done with Activated Sludge Process while the Secondary-stage involved Electro Contaminant Removal. Finally, the effluents from the biometric CETP were compared with the Bangladesh Standards for wastewater quality discharging into various receiving bodies. The Environment Conservation Rules (ECR, 1997) of the country has set a range of water quality standards for different receiving bodies such as Inland Surface Water, Public Sewer, and Irrigated Land. In biometric CETP, the removal efficiency of BOD was found to be 97.40% while the Dissolved silica, Clays, Carbon black and Suspended materials in water were removed generally around 98%. The Treatment Plant reduced Bacteria, E-coli and Viruses almost completely. Emulsified oily substances as well as heavy metals in water such as arsenic, cadmium, chromium, lead, nickel, and zinc were removed in the range of 95 to 99%. Both the primary and secondary stages of treatment operations were found to have significant contribution in the overall treatment performance of the biometric CETP upholding the final effluent quality within the permissible limits of ECR, 1997.

Keywords: Bio-Electric treatment, Central Effluent Treatment Plant, Co-mingled industrial wastewater, performance evaluation

4. INTRODUCTION

Water is the basic need of all life, human, well-being and also for economic development. Because of increasing industrialization, urbanization and other anthropogenic activities, and the water quality is getting degraded day by day. Waste water management is a concept involving several operations, which depend on the scale and nature of industry or institution and end use of wastewater. The objective of wastewater management at the present scenario is to solve the environmental pollution problems and water scarcity by adopting reducing, reusing or recharging methodologies. At present the conventional methods of wastewater treatment are working with a limited treatment efficiency to comply with the requirements of the regulatory bodies. Besides it is difficult for each industrial unit to provide and operate individual wastewater treatment plant because of the scale of operations or lack of space or technical manpower. However, the quantum of pollutants emitted by small scale industries may be more than an equivalent large-scale industry. Common Effluent Treatment Plant is the concept of treating effluents by means of a collective effort mainly for a cluster of small scale industrial units. The main objective of CETP is to reduce the treatment cost for individual units while protecting the environment and to achieve economical waste treatment, thereby reducing the cost of pollution abatement for individual factory [CETP,2009]. However the effectiveness of the wastewater management will depend on the treatment methodology, operation and maintenance of the equipment and the commitment of operation/management personnel. Prevention of pollution of natural resources such as land and water by the waste water and adequate preparation or renovation of the wastewater before reuse, are further important considerations in formulating and designing appropriate waste water disposal arrangements.

The degree of treatment provided to the wastewater will largely be based on the effluent standards prescribed by the regulatory agencies when the treated effluent is to be discharged into a watercourse or land. If the effluent is to be reused, the quality of the effluent required to support such reuse will indicate the degree of treatment necessary. The complete treatment of wastewater is brought by a sequential combination of various physical unit operations, and chemical and biological unit processes. The general yardstick of evaluating the performance of sewage treatment plant is the degree of reduction of BOD, and suspended solids, which constitute organic pollution.

Performance evaluation of existing treatment plant is required to assess the existing effluent quality and/or to meet higher treatment requirements and, to know about the treatment plant whether it is possible to handle higher hydraulic and organic loadings. Performance appraisal practice of existing treatment plant units is effective in generation of additional data which also can be used in the improvement in the design procedures to be followed for design of these units. Existing facilities can be made to handle higher hydraulic and organic loads by process modifications, whereas meeting higher treatment requirements usually requires significant expansion and/or modification of existing facilities (USEPA, 1974).

One of the primary considerations in evaluating an existing wastewater plant is in the area of plant operation and control, proper process control in frequent and accurate sampling and laboratory analysis. Poor conditions of sewerage system, improper design of the plant and organizational problems are important factors that cause treatment plant not to meet the effluent standards (Storhaug, 1990). Overloading due to increase in population and water use, discharge of trade effluents are other reasons of recent times for the poor performance of waste water treatment plants .The treatment efficiency may be badly affected if the system is hydraulically under loaded (Kapur, Prasad and Gupta,1999).An export processing zone (EPZ) is defined as a territorial or economic enclave in which goods may be imported and manufactured and reshipped with a reduction in duties / and/or minimal intervention by custom officials (World Bank ,1999). DEPZ (Dhaka Export Processing Zone) is situated in Savar. 35 kms from Dhaka city centre, 25 kms from Hazrat Shah Jalal (R) Airport, 304 Kms from Chittagong Sea Port. Because of this suitable location, As on December, 2010, a total of 94 industries have invested 785.93 million US dollars in here, and by 2012-13 (February, 2013) it has risen to 911.64 million US dollars (BEPZA ,2009).

Figure 1: Google image of DEPZ Central Effluent Treatment Plant [C.E.T.P]

To ensure safe water and environment for the inhabitants of Savar and outskirts, by way of protecting biodiversity, natural resources and water bodies, a biometric central effluent treatment plant is constructed on a stretch of 16,000-square-foot area of DEPZ old zone and it started its treatment in February’ 2012. It is the first of its kind in Bangladesh and a first in the world to utilize a Bio-Electric treatment profile on a co-mingled industrial wastewater stream. The Plant possesses the best and latest combinations of technology and equipment provided and manufactured by Singapore company Flagship Ecosystems Investment Private Limited (FESI). It has the capacity to treat 43, 000 cubic meters of water and 15,000 cubic meters of ECC from DoE per day. The recycled water will be poured into nearby Dholai Beel. The main objectives of this study is to analyze the overall performance of the Central Effluent Treatment Plant and the removal rates of various parameters like pH, BOD, COD, TSS and TDS as well as performance and removal efficiencies of the individual units in the primary and secondary treatment.

5. METHODOLOGY

Waste water samples were collected at different stages of treatment units and analyzed as outlined in the standard methods for the examination of water and wastewater (APHA,1998). The samples were analyzed for various parameters like pH, BOD, COD, TSS and TDS and compared with Bangladesh National Standards for Waste Discharge Quality according to ECR 1997(The Environment Conservation Rules) for Inland Surface Water, Public Sewer, and Irrigated Land (MoEF,2008).Reduction Capabilities and Removal Rates of several other parameters in Primary Stage (Activated Sludge Plant) and by Secondary (ECR) Treatment were also analyzed.

5.1 COMMISSIONING PROCEDURES OF TREATMENT

Waste water gets diverted to CETP. All sluice gates are closed. Tank 2 (Figure.2) is allowed to fill and overflow to by-pass channel. The integrity of Tank 2 is checked and Drum Screens are powered up. Sluice gate is opened and integrity of Tank 4 is checked. Drum Screen rotation is started. Main Lift Pumps start up sequentially 1,2,3,4. Pump 1 fills Tank 8B. Pump 2 fills Tank 6. Pump 4 fills Tank 8A.When the height of 6 reaches 1/3 of tank height pumps will be shut down (i.e. 2&3). When 8A/8B reaches 1/3 height all pumps will be shut down. The integrity of Tank 6, 8A/8B are checked. It is held for 6 hours and Drum screens are shut down. After 6 hours all pumps are started up to fill 6, 8A/8B to full overflow level. When 7A/7B is full shut all pumps and integrity is checked. All 4 main pumps are re-routed to Tank 6. Pumps are started. Overflow to Clarifier Tank 10 is done just before to Tank 11. Pumps are shut and integrity is checked. Then pumps are started to overflow Tank 11. Pumps are stopped when Tank 11 is full and integrity is checked. Then ECR feed pumps are started. Overflowed to Tank through 13 and held. Integrity is checked. Tank 13 is allowed to overflow. Clarifier Tank 12 until overflow to Tank 14. Pumping is started from Tank 12 bottom to fill Tanks 16/17/18. Integrity is checked. Then it is held for 24 hours and full inspection is conducted on all Tanks integrity for leaks, cracks and other faults.

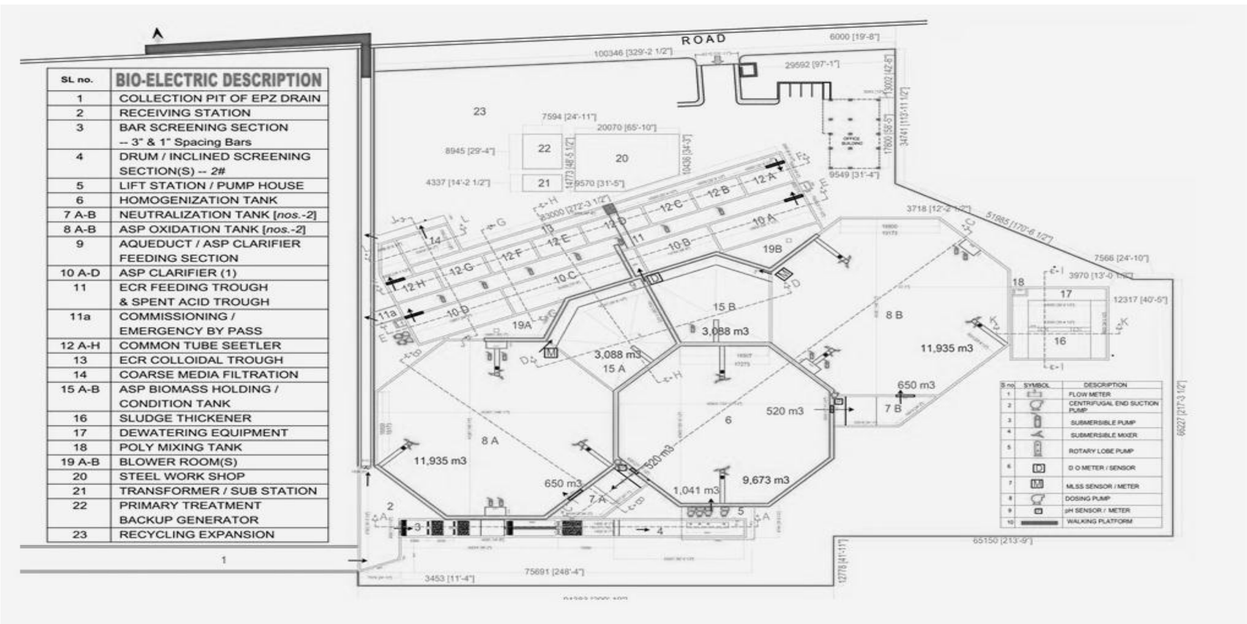

Fig 2: DEPZ Central Effluent Treatment Plant Detail Layout.

5.2 TREATMENT PROCEDURES

The treatment process can be divided in two steps: - Activated Sludge Plant (ASP)-Primary Treatment and Electro Contaminant Removal (ECR)- Secondary Treatment. The process is shown in a flow chart below:

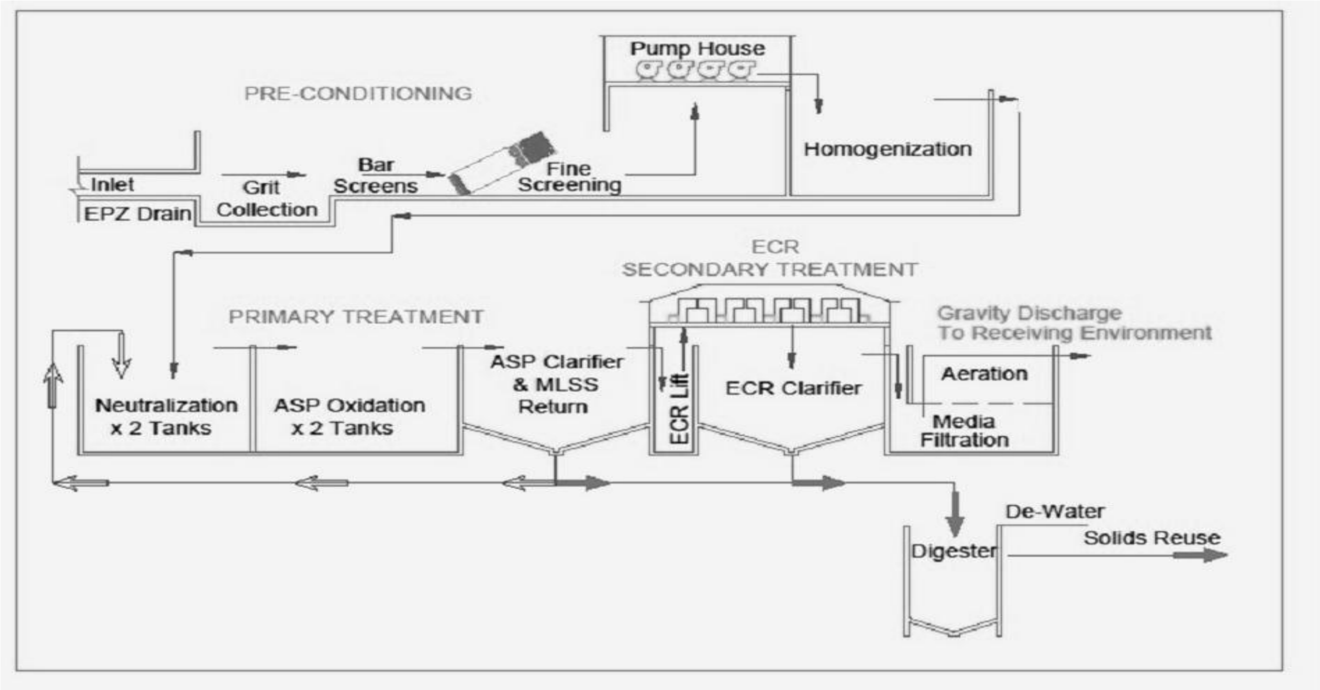

Figure 3: Process Flow Diagram of Central Effluent Treatment Plant at Dhaka EPZ.

6. RESULTS AND DISCUSSION

The evaluation of performance efficiency of the plant was undertaken in terms of effluent quality. The evaluation was based on the plant operation data such as pH, Bio- chemical Oxygen Demand(BOD), Chemical Oxygen Demand (COD), Total Suspended Solids, Total Dissolved Solids, measurements for the period of 5 weeks from February'13 to March'13 . The variations of these parameters during the study are shown graphically in Fig 4, Fig 5, Fig 6, Fig 7 and Fig 8. It has been observed that the overall treatment efficiency for the BOD removal is 97.40% and that for Total Suspended Solids removal is 98%.

Figure 4 shows the variation of the values of pH of the collected samples as well as their average value and the national standard foe pH according to ECR 1997. The samples have an average pH of 7.39. The ECR 1997 standard of pH for Inland Surface Water, Public Sewer, Irrigated Land is 6-9.

Figure 5 shows the variation of the BOD values having an average of 44.5 mg/l. The national (ECR 1997) standard for BOD for Inland Surface Water is 50 mg/l, Public Sewers 250 mg/l, Irrigated Land is 100 mg/l which is also shown in the figure.

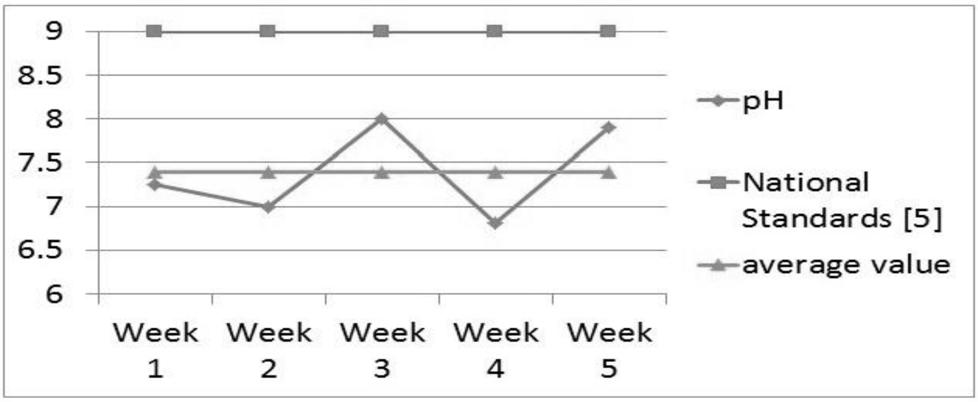

Figure 4: Variations of pH

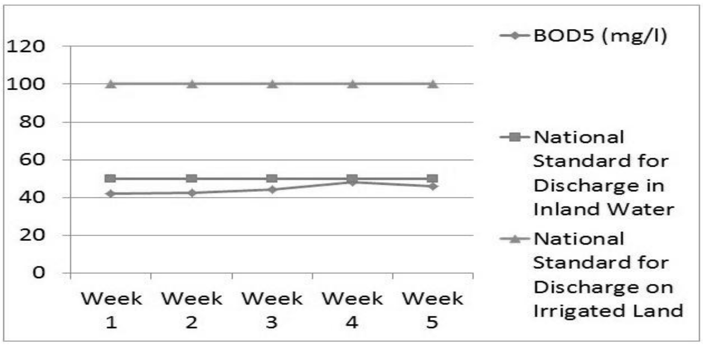

Figure 5: Variations of BOD

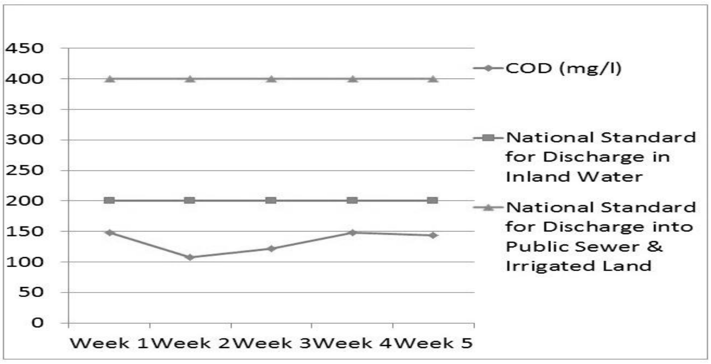

Figure 6: Variations of COD

Figure 6 shows the variation of the COD values (average of 134 mg/l).It also shows the national (ECR 1997) standard for COD values for Inland Surface Water which is 200 mg/l & for Public Sewers and Irrigated Land a value of 400 mg/l. Figure 7 & Figure 8 shows the variation of the TSS and TDS values in the collected samples respectively. They also show the national standard for these parameters according to ECR 1997.

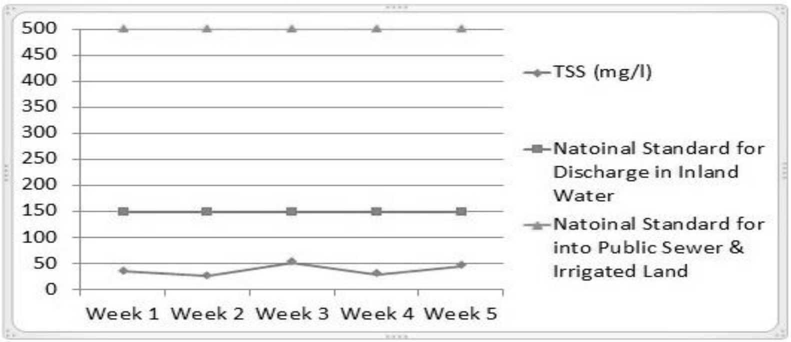

Figure 7: Variations of TSS

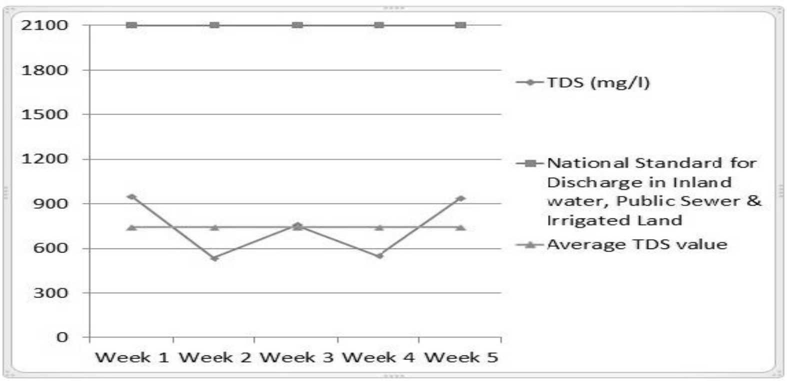

Figure 8: Variations of TDS

DEPZ Central Efflunt Treatment Plant has a sequential contaminant removal process. After the pre-conditioning and equalization the primary treatment starts in the Activated Sludge Plant (ASP) which consists of Neutralization – Feeding Section, Oxidation tank, Clarifier, and Sludge retaining -conditioning sections. Table 1 shows in detail the reduction capabilities and removal rates (average values)of the various parameters present in the effluent in Primary Stage

Table 1: Reduction Capabilities and Removal Rates in Primary Stage/ Activated Sludge Plant (ASP)

	Parameters	***Specifications***	***ECR 1997***	***Remarks***
	Wastewater flow rate	1750 m^3/hour		24 hours CETP operations
	Inherent Fibrous material and Lint	2mm removal@ 75%		Automated fine screen
RAW Influent MIX parameters and combined bio & electric reduction values				***Bio – Electric removal %***
	Color	300 Pt Co unit		90% removal by ECR
Raw	Chemical Oxygen Demand (COD) [industrial-sewerage-rinse waters]	500-700 mg/l		Assumed total co-mingled streams
Raw	Biochemical Oxygen Demand (BOD)	166-230 mg/l		Assumed to be 1/3 of COD value
Pri	Post bio before ECR COD	250-350mg/l		60% COD reduction with aeration by ASP
Pri	Post ECR after clarification COD	100-140mg/l	200	Minimum COD reduction up to 60% by ECR
See	Post bio before ECR BOD	83-115 mg/l		60% BOD reduction with aeration by
See	Post ECR after clarification BOD	33-46 mg/l	50	Minimum BOD reduction up to 60% by ECR
Raw	Total Suspended Solids (TSS)	350 mg/l	150	TSS- 90% reduction with secondary separation techniques/Clarifier
Raw	Total Dissolved Solids (TDS)	Average 1500 mg/l	2100	Average 1500 mg/l
	pH	Average 7.5 – 8	6-9	Adjusted by bio/ASP activity to 7.5-8
Raw	Dissolved Oxygen (DO)	1.5-3.0 mg/l		With minor sewage instruction
	DO	4.5-5.0 mg/l	4.5	Combined EQ & ASP Oxidation
	Ammonia Nitrate	<50 mg/l		By ASP & ECR
	Arsenic	< .2		96% other heavy metals removal by ECR

The secondary process is Electro Contaminant Removal (ECR). It is a process of destabilizing suspended, emulsified or dissolved contaminants in an aqueous medium by introducing an electrical current into the bio water stream. Table 2 shows the removal rates (average values) of the various parameters by Secondary (ECR) Treatment

Table 2: Removal Rates by Secondary (Electro Contaminant Removal) Treatment`

Heavy metals	Average % Removed	Other contaminants	Average % Removed
Aluminum	99	Aldrin	98
Arsenic	96	Chloreiviphos	99
Barium	98	DDT	99
Calcium	98	Diazinon	99
Iron	99	Cyanide	99
Lead	97	Nitrate	40
Magnesium	98	Linade	99
Manganese	83	Boron	70
Mercury	66	E- Benzene	99
Nickel	99	Hydrocarbons	98
Zinc	99		

The Equalization Tank and ASP is offered with sufficient retention time, so that any fluctuations in flow and pollutant concentrations can be homogenized with constant gravity feed to biological system. A constant, homogenized feed is essential for the proper maintenance of the biological system and microbial mass. This also helps in reducing the temperature of the composite effluent for effective microbial lifespan. Electro Contaminant Removal (ECR) System offers tremendous advantages. Operating costs are low compared with chemically enhanced treatment. ECR provides substantial difference (as much as 85% less) in sludge generation as compared to chemical usage. It has positive removal impact on other contaminants that may be part of total (co-mingled) waste waters entering CETP from DEPZ factories. It also avoids the use of chemicals and so there is no problem of neutralizing with excess chemicals and no possibility of secondary pollution caused by chemical substances added at high concentrations.

7. CONCLUSIONS

The overall performance of the Central Effluent Treatment Plant was satisfactory. The removal efficiency of BOD was found to be 97.40% and that of Dissolved silica, clays, Carbon black, and other suspended materials in water are generally reduced by 98%. The Treatment Plant reduced bacteria, E-coli, and Viruses. Emulsified oily waste waters were reduced by 95 to 99%. Heavy metals in water such as arsenic, cadmium, chromium, lead, nickel, and zinc are generally reduced by 95 to 99%. The individual units were also performing well and their removal efficiencies were satisfactory.

REFERENCES

APHA (1998). AWWA, WPCF. Standard Methods for the Examination of Water and Wastewater. 20th edition, American Public Health Association, Washington, DC, New York, USA, 1998.

BEPZA (2009). Bangladesh Export Processing Zones Authority: http://www.epzbangladesh.org.bd/bepza.php?id=YRINV (Accessed on 17/08/2013)

CETP2009. Common Effluent Treatment Plant: A solution or a problem in itself, Toxics link, November 2000; http://www.toxicslink.org/docs/06038_CETP_Report.pdf

EPA (1974). Environmental Protection Agency, US, *Process design manual for upgrading existing waste water treatment plants*, Technology Transfer,13-17.

Kapur, A., Kansal, A., Prasad, R. K. and Gupta, S (1999). Performance evaluation of Sewage Treatment Plant and Sludge bio- methanation, Indian Journal of Environmental Protection, 19. 96 - 100.

MoEF (2008). Guide for Assessment of Effluent Treatment Plants, Department of Environment, Ministry of Environment and Forest, Bangladesh, First Edition, 4-6

Storhaug, R. (1990). Performance stability of small biological chemical treatment plants, Water Science and Technology, 22. 275-282.

HOUSEHOLD LEVEL PURIFICATION TECHNIQUE TREND IN DHAKA CITY

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ABSTRACT

As Dhaka is the capital of Bangladesh, rapid urbanization and population growth in last decades have changed the physical environment of this city. An estimated 3.4 million people live in 5000 slums of Dhaka (Islam, 1999).The population of Dhaka city is growing so fast that the improvements on water supply have failed to keep pace. This study concentrated to provide an overview of drinking and non-drinking water source pattern of various socio-economic groups .According to this study, 71% city dwellers depend on Dhaka Water Supply and Sewerage Authority (DWASA) supplied water sources (both piped supply and deep tube well) for drinking purpose .The study areas were selected on the basis of vulnerability to water borne diseases .This study also focused on pattern of purification technique used at user's end and co-related with socio-economic aspect. In this study it was observed that about 39% peoples used no purification method at user's level of the study areas which leads to a very alarming situation.

Keywords: Dhaka City , Drinking Water, Purification Technique ,User's End,

1. INTRODUCTION

The rapid and uneven urbanization has resulted in growth of informal settlements, inter-city and intra-city gaps in water supply coverage. About 35 percent slum dwellers of Dhaka city is living in only 4 percent of land area while the hygienic sanitation in slums is only 12 percent at the moment. Piped water coverage in Dhaka is about 83 percent. The gap is wide in the consumption pattern as from about 20 litres per capita-day in low income slums to about 400 litres per capita-day are provided in high income areas. Rapid urbanization is posing a great problem to ensure potable water supply for all in Dhaka City. Only 55% of the Dhaka's urban poor currently receive tap water (Siddiqui et al, 2004), and less than 70% of its slum dwellers have access to safe drinking water (Chowdhury, 1999). As a result of a severe lack of access to safe drinking water in these areas, residents experience an unsanitary lifestyle and suffer from increased health risks. Most of the municipalities and service agencies are unable to cope with the pressures of providing basic services to this expanding population; due to limited resources and an excessive demand for water within the service area. As a result Dhaka's urban poor have very little involvement with utility services (Podymow et al, 2010). Water related diseases are responsible for 24% of all deaths and gastroenteritis and diarrheal diseases kill 110,000 children below the age of five every year (Water Aid, 2011). This study explored drinking and non-drinking water sources of various socio-economic groups and co-related it with socio-economic aspect. This study also evaluated consumers' satisfaction regarding drinking water quality. This study identified various household level purification technique and their corresponding users. Effort has been made to collect relevant data from the concerned study groups by conducting questionnaire survey and field inspection of sources of non-drinking and drinking water.

2. METHODOLOGY

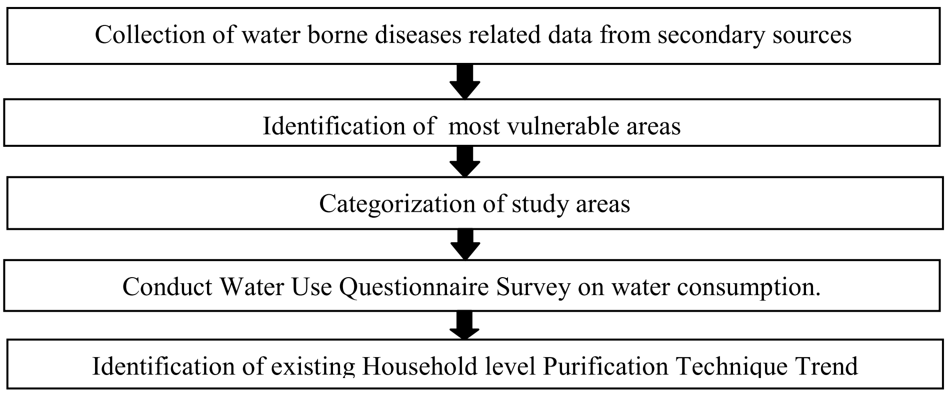

Figure 1: A flow diagram of methodology

2.1 Basis of Selection of Study Areas

Study areas were selected based on following criteria-

- Socio-economic condition.
- Vulnerability in terms of water quality (microbial).
- Amount of drinking water consumption.
- Sensitivity.

2.2 Socio-economic condition

Socio-economic aspect of user is an important factor for the assessment of drinking water source and purification method. The selection of source of drinking water and purification method is greatly influenced by user's –

- Income.
- Perception regarding health impact of using contaminated water.
- Willingness to pay for drinking water.

2.3 Vulnerability in terms of water quality (microbial)

The areas of Dhaka city where people are most vulnerable to water-borne diseases were selected based on International Centre for Diarrhoeal Disease Research, Bangladesh (ICDDR,B) report.

2.4 Amount of drinking water consumption

Amount of water consumed for drinking purpose varies from group to group. This amount depends on-

- Number of member of the group: A group having more members apparently consume more water than a small group.
- Water Consumption period.

In this study, study areas were grouped into three categories depending on the amount of drinking water consumption as found in field survey:

- Low: Amount of drinking water consumption is less than 50 lpd.
- Medium: Amount of drinking water consumption is greater than 50 lpd but less than 500 lpd.
- High: Amount of drinking water consumption is greater than 500 lpd.

Classification of study group based on drinking water consumption is shown in Table 1.

Table 1: Classification of study group based on drinking water consumption

	No. of sample	Remarks
Low	29	All of Residential settled and slum areas were in this group.
Medium	5	All of offices were in this group.
High	18	All of schools, colleges, hospitals and markets were in this group.

2.4.1 Sensitivity

Here the term "**sensitivity**" refers to the degree of health impact on a group caused by drinking water quality. According to sensitivity , study areas were classified into two categories-

- Low Sensitive Group: Residential areas (both settled & slum)
- High sensitive Group: School/College, Hospital etc.

3. DATA ANALYSIS

3.1 Non-Drinking Water Sources for Dhaka City

This study has revealed that city dwellers use mainly two types of water sources for household purpose:

- DWASA (Deep Tube Well)

- DWASA (Piped Supply)

  Only a few households were found using rain water.

Figure 2 represents a distribution of non-drinking water sources of the study area. This study revealed that about 60% of respondents were satisfied regarding the quality of non-drinking water.

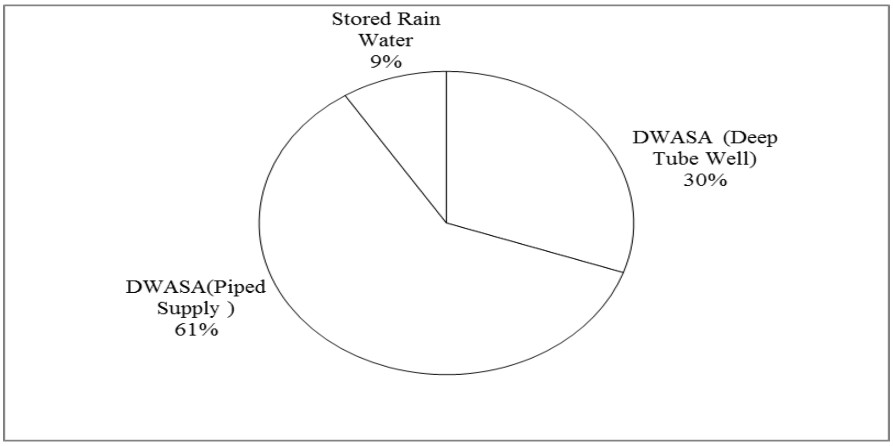

Figure 2: Non-drinking Water Source title

3.2 Sources of Drinking Water for Dhaka City

Five types of sources are used for drinking purpose in this city as per this study:

- Bottled Water
- DWASA (Deep Tube Well)
- DWASA(Piped Supply)
- Water Jar with Dispenser
- Stored Rain Water

Figure 3 presents a bar diagram of area wise distribution of various sources of drinking water of the study sample. From this figure it is clear that the commercial sectors like offices and markets are entirely dependent on private sector initiated water jar with dispenser for drinking purpose. On the other hand slum and residential settled areas fully depend on DWASA initiated pipe supply or deep tube well.

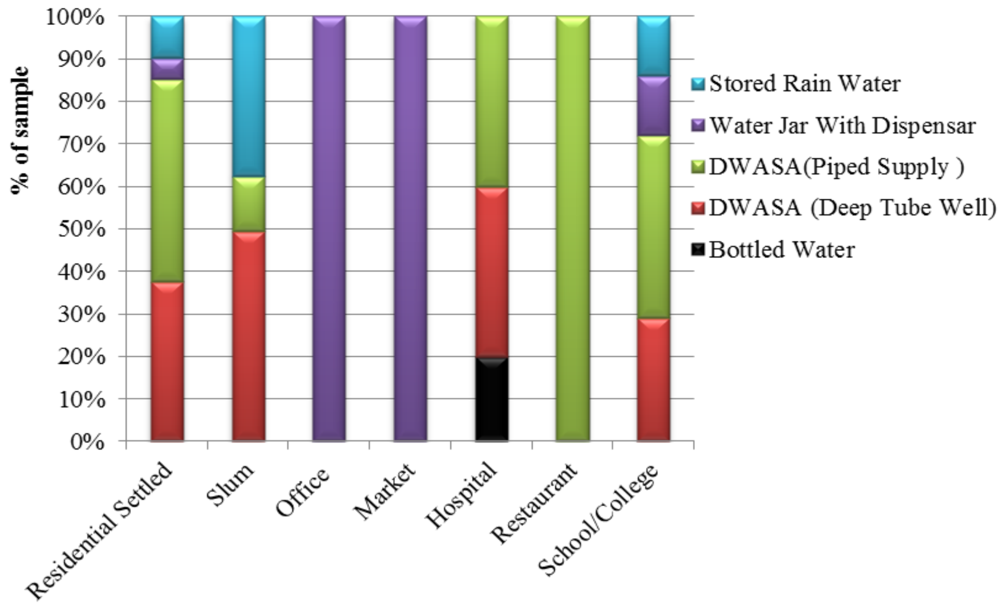

Figure 3:Group wise pattern of source of drinking water.

3.3 Group Wise Pattern of Household Level Purification Techniques

In this study three types of disinfection systems are found to be used at user level such as boiling, filtration and combination of boiling and filtration.

3.3.1 Residential Settled

This study groups mainly use boiling and a combination of boiling and filtration as purification system. Figure 4 manifests the proportion of various purification systems used by this study group.

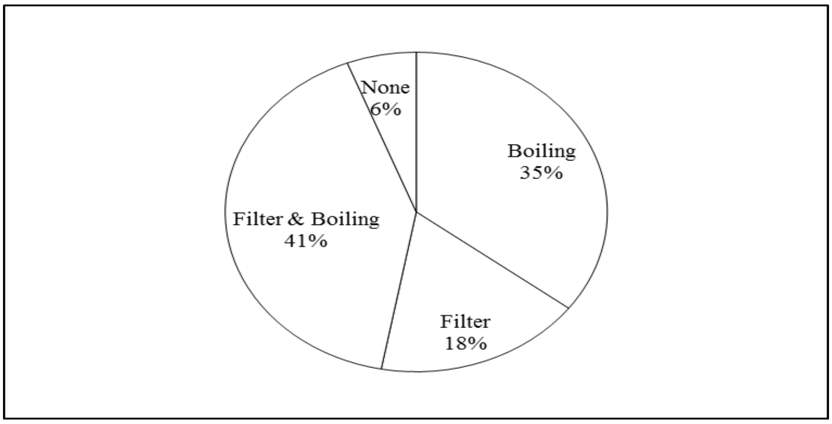

Figure 4: Proportion of purification systems currently used by consumers of Residential Settled Areas.

3.3.2 Slum

This group is considered as the most vulnerable for water borne diseases. This group can be classified as-(Reference–Section 2.1)

- Low Income Group
- Low water consumption Group
- Low sensitive Group

Figure 5 shows that only ***11***% users use filtration for disinfecting drinking water and rest ***89***% use no form of purification system. Compared to the Residential Settled areas in slum areas people are less likely to use purification method .

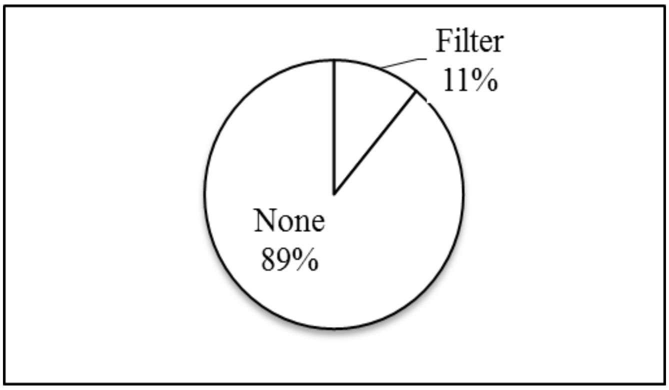

Figure 5: Proportion of purification systems currently used by consumers of Slum Areas

3.3.3 School/College

This group is sensitive from the view point of water consumption as a large number of people is served simultaneously at these institutions and more importantly most of the consumers of this groups are children or age below 18 years. For this reason this group draws special attention in the assessment of drinking water quality. Figure 6 indicates that about ***72%*** school and college of the study sample do not use any form of purification system .

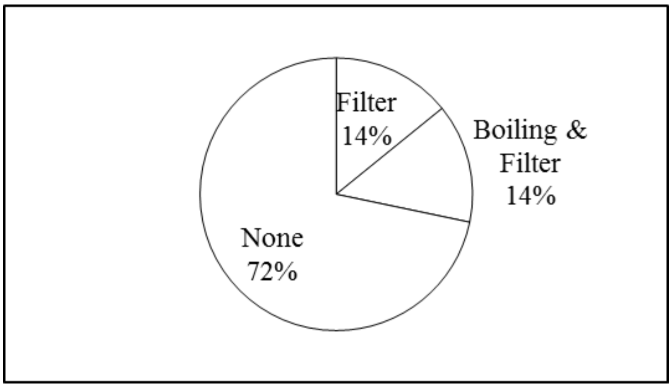

Figure 6: Proportion of purification systems currently used in schools and colleges.

3.3.4 Market

From questionnaire surveys conducted during this study it was revealed that all of the shop owners and their workers of market places rely on water jar with dispenser for drinking purpose in the study areas. These people are hardly conscious about the quality of the drinking water and according to this study 33% of them do not use any purification system while rest 67% use filtration technique.

3.3.5 Restaurant

Restaurants are highly sensitive from the perspective of drinking water quality due to involvement of many people. All of the restaurants considered in this study were found using filtration as their purification system.

3.3.6 Office

Offices of various banks and private firms are considered in this study. Employees usually work in these places for eight hours or more. So, the quality of drinking water of these places exerts an important dimension on the health of employees. This study identified that all of the offices considered in this study were reported to use Water Jar with dispensar as drinking water source in most cases (***80%***) without implementing any purification techniques. Following Figure 7 shows the trend of using various purification techniques in offices.

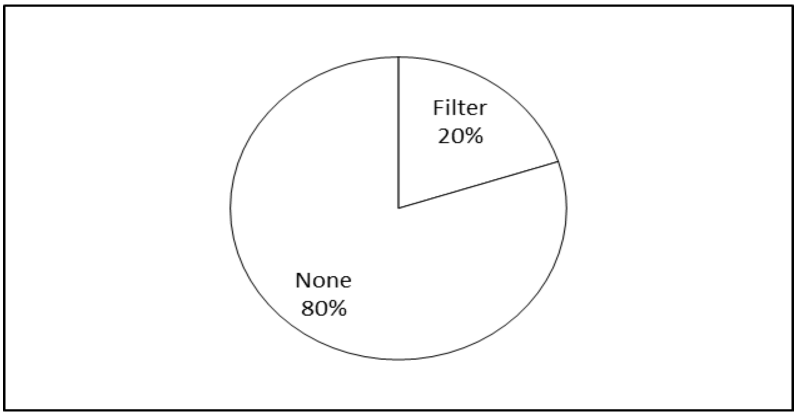

Figure 7: Proportion of purification systems currently used by consumers of Offices.

3.3.7 Hospitals

Both private and public hospitals were taken account in this study .This study area is highly sensitive from the drinking water point of view as a large number of vulnerable patients are involved in this study groups. In some cases patients use bottled water specially for public hospitals and in other cases patients use water supplied by the hospital authority for drinking purpose. This study reveals that ***80%***hospitals use filtration for sterilization (Figure-8).

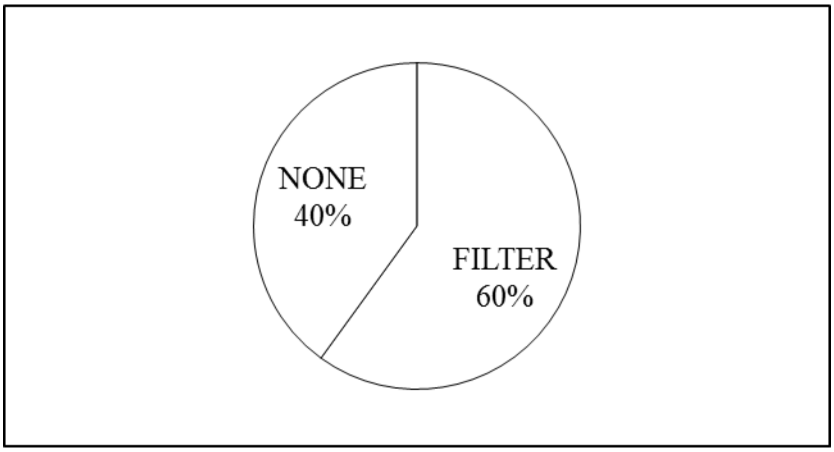

Figure 8: Proportion of purification systems currently used in Hospitals.

4. CONCLUSIONS

From the group wise pattern of drinking water source, it was apparent that offices and markets were fully dependent on private sector that promoted water jar with dispensar but dwellers of residential settled and slum areas mainly used government initiated DWASA supplied water as their drinking water source. As per finding of this study, about 19 % people used both boiling and filter machine,14% people used only boiling and 28% people used only filter machine and 39% people used no purification method at user's end. So it is noticeable that a large portion of population of the study areas were not using any sort of purification techniques. This fact was more applicable for the slum dwellers as this study found that 89% of them did not use any form of purification techniques. As a result they are more vulnerable to water-borne diseases.

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REFERENCES

Ahmed, M.F.and Rahman, M.M. (2000). Water Supply and Sanitation – Rural and Low Income Urban Communities. International Training Network (ITN) Centre, BUET.

Alam, M.J. (2007). "Water Quality Tests and Behavioral Factors of Child Diarrhoea in Dhaka slums." BRAC University Journal, Vol. IV(1), 103- 109.

Chowdhury, N. T. (1999). Willingness to Pay for Water in Dhaka Slum: A Contingent Valuation Study. IUCN.

Islam, M.R.(1999) "Drinking Water Quality and Sanitation Condition of Flood Shelters in Dhaka City", M.Sc. thesis, Department of Civil Engineering, Bangladesh University of Engineering and Technology (BUET).

McGhee, T.J.(1991). Water Supply and Sewerage. McGraw-Hill, Inc, New York, USA.

Peavy,H.S.,Rowe,R.D.and Tchobanoglous,G.(1985). Environmental Engineering. McGraw-Hill, Inc, New York, USA.

Podymow, T., Turnbull, J., Islam, M.A. and Ahmed M. (2010). "Health and Social condition of Dhaka Slums", http://www.isuh.org/download/dhaka.pdf.

Siddiqui, K., Ghosh A, Bhowmik S. K., Siddiqi S., Mitra M., Kapuria S., Ranjan N., and Ahmed J. (2004) Megacity Governance in South Asia: A Comparative Study. University Press Limited, Dhaka, Bangladesh.

Water Aid in Bangladesh (July 2011). Bangladesh Country Strategy Plan: 2011-2016, Banani, Dhaka, Bangladesh.

WHO (2007). Guidelines for Drinking-water Quality. Report of World Health Organization, World Health Organization, Switzerland.

IMPACT ASSESSMENT OF CYCLONIC STORM SURGES ON ECOYSTEM SERVICES IN THE SOUTHWEST COASTAL ZONE OF BANGLADESH

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ABSTRACT

Two of the most devastating cyclones experienced lately by Bangladesh are Sidr and Aila, both of which hit the southwest coast of Bangladesh on November 15, 2007 and May 25, 2009 respectively. Both the cyclones slammed the coastal areas of Bangladesh and several rivers broke through embankments, causing widespread flooding. It took several months to repair these broken embankments and draining out the excess rain & surge waters trapped inside the polders. This caused long-term flooding which affected the ecosystem services in the coastal area. This paper explores the temporal variation of wetlands before & after the two cyclonic events and its impact on ecosystem services of coastal zone. The study area covers the southwest coastal zone of Bangladesh, which includes the Satkhira, Khulna and Bagerhat districts. Important ecosystem services in the study area include agricultural lands and wetlands which support fisheries resources. To assess the impact of cyclonic storm surges in the study area, four Landsat scenes of 2007, 2008, 2009 and 2010 are analyzed using ILWIS. Supervised land cover classification has been conducted using maximum likelihood classification algorithm. Land cover classes included agricultural lands and wetlands. Secondary data on crop and fisheries production has been collected from BBS.

Keywords: Ecosystem services, storm surge, coastal zone, land cover classification and remote sensing

1. INTRODUCTION

The coastal area of Bangladesh is extremely vulnerable to cyclone-induced storm surge. In fact, UNDP had identified Bangladesh to be the most vulnerable country in the world to tropical cyclones (UNDP, 2004). There are mainly three reasons behind it. The first one is that the continental shelf is long and shallow and the funnel shape of the coast tends to concentrate and amplify the surge in the northern part of the Bay. Secondly the coastal zone is low-lying with 62% of the land having an elevation of up to 3 meters and 86% up to 5 meters from the mean sea level (IWM, 2009). The third reason is that the coastal area is densely populated, accommodating about 50 million people, nearly one-third of the total population of Bangladesh (Miyan, 2009).

About one-tenth of the global total cyclones forming in different regions of the tropics occur in the Bay of Bengal. About one-sixth of the tropical storms generated in the Bay of Bengal usually hit the Bangladesh coast. Historical record shows that more than 15 severe cyclones are generated in the Bay of Bengal in every ten years. During 1960-2007 about 18 severe cyclones hit the coast of Bangladesh (IWM, 2009). During the period 1582 to 1997 there were 82 cyclones that devastated the coastline of Bangladesh (Jacobson et al. 2006). These cyclones originated mainly in Indian Ocean or Bay of Bengal generally form in the months just before and after the monsoon and typically move to north to northeastern direction towards the land (SMRC, 1998). It is seen that the eastern coast experiences maximum inundation between 4-6 m and western coast experiences inundation within the range of 3-5 m (IWM, 2009).

Two of the most devastating cyclones experienced lately by Bangladesh are Sidr and Aila, both of which hit the southwest region of Bangladesh on November 15, 2007 and May 25, 2009 respectively. Cyclone Sidr formed in the central Bay of Bengal, and quickly strengthened to reach peak 1-minute sustained winds of 260 km/h (160 mph), making it a Category-5 equivalent tropical cyclone on the Saffir-Simpson Scale. It slammed the coastal areas of Bangladesh with heavy rain, strong winds and a storm surge. At least 3,447 deaths were caused by the storm. Again, Cyclone Aila with a storm surge of 3 m (10 ft) impacted the western coastal zone of Bangladesh, submerging numerous villages. Several rivers broke through embankments, causing widespread inland flooding. Torrential rains from Aila resulted in at least 190 fatalities from flooding (DMB, 2009). It took several months to repair these broken embankments as well as draining out the excess rain & surge waters trapped inside the polders. These caused long-term water-logging and inland flooding during high tides which affected the ecosystem services in the coastal area.

Ecosystems form the life-supporting system of the earth. Ecosystems are very important as they provide the basis for human civilization and natural capital for green economy and sustainable development. Ecosystems provide four types of potential services such as supporting services, regulating services, provisioning services and cultural services. Ecosystem services are components of nature. Ecosystem components include resources such as surface water, oceans, vegetation types and species. Ecosystem process and functions are the biological, chemical and physical interactions between ecosystem components (Boyd and Banzhaf, 2007). Ecosystem services may range from crops, fish, freshwater to those that are harder to see such as erosion regulation, carbon sequestration, and pest control. Ecosystem services are getting attention to the environmental community for conservation missions because the protection of ecosystem services is vital in securing the long-term livelihood of people.

The southwest coastal zone of Bangladesh consists of Satkhira, Khulna and Bagerhat districts. Important ecosystem services in this area include agricultural lands and wetlands which support fisheries resources and shrimp farming. The agricultural sector of Bangladesh contributes to 17.5% of GDP (2012 est.; CIA, 2013). It dominates both land use and the national economy. It supports people's lives and livelihood in this region. The performance of agricultural production has an overwhelming impact on major macroeconomic objectives like employment generation, poverty alleviation, human resources development and food security. However, The farmers in Satkhira-Bagerhat-Khulna area have switched over to shrimp culture from traditional agriculture, allowing more and more salt-water in the land. There are 37,400 bagda (saltwater shrimp) fields with an operated area of 170000 ha (PDO-ICZMP, 2003). A large proportion of delta populations experience extremes of poverty and are highly vulnerable to the environmental and ecological stress and degradation that is occurring (ESPA-Deltas, 2013). Certain changes (i.e. adaptation) are necessary as people in the coastal zone of Bangladesh face multiple threats (extreme poverty, food insecurity due to population increase, sea level rise, salinization, cyclones, etc.). In this context, this paper explores the variation of wetlands before & after the two major cyclonic events Sidr and Aila, using remote sensing techniques and, the subsequent effects on ecosystem services in the western coastal zone of the country.

2. STUDY AREA

The study area is located in the southwest coastal zone of Bangladesh. It includes three districts of Khulna division – Satkhira, Khulna and Bagerhat. The study area lies between latitudes 21o39´00´N to 23o05´00´N and longitudes 88o54´00´E to 90o00´00´E. The Sundarbans, world's largest mangrove forest is the southern part of the area. Khulna district has an area of 4394 km². The total population of Khulna district is 2,318,527. It is one of the important industrial and commercial areas of the country. 75% of shrimp exported from Bangladesh are cultivated in the Khulna zone. It is also known for its lobster, prawn, catfish, and crab. Satkhira district has an area of 3858 km². The total population of Satkhira district is 1,843,194. Most of the peoples of southern part of Satkhira depend on pisciculture. The main exports are shrimp, paddy, jute, wheat, betel leaf, leather and jute goods. Bagerhat district has a total area of 3959 km². The total population of Bagerhat district is 1,476,090 (BBS, 2012). Rampal and Fakirhat - two Upazillas of Bagerhat, is known for its huge production of shrimp.

3. METHODOLOGY

3.1 Data Collection

The study used satellite images downloaded from webpage of United Sates Geological Survey (http://www.earthexplorer.usgs.gov) and secondary data collected from national agencies such as Bangladesh Bureau of Statistics and Department of Fisheries, Bangladesh. In order to assess the impact of cyclonic storm surges in the study area, four Landsat scenes of 2007, 2008, 2009 and 2010 are analyzed using ILWIS 3.4. Supervised landcover classification has been performed using maximum likelihood classification algorithm for 7,4,2 Landsat TM band combination. Landcover classes include agricultural lands and wetlands. The images represent dry season of Bangladesh as all of them have been captured in the month of January. The images have been These images are taken at four tiles: Path of 137 with Row of 44, Path of 137 with Row of 45, Path of 138 with Row of 44 and Path of 138 with Row of 45, which cover the entire study area. Details of the images are presented in Table 1.

Table 1: Details of Landsat satellite images

Image No.	Acquisition Date	Satellite	Sensor	Spatial Resolution (m)
1	January, 2007	Landsat 5	TM	30
2	January, 2008	Landsat 5	TM	30
3	January, 2009	Landsat 5	TM	30
4	January, 2010	Landsat 5	TM	30

Secondary data include crop, fish and shrimp production at the study area for fiscal years 2006-07, 2007-08, 2008-09 and 2009-10 which are later compared with image analysis results for 2007, 2008, 2009 and 2010 respectively.

3.2 Satellite Image Processing and Land Cover Classification

Integrated Land and Water Information System (ILWIS, 2012) has been used to process the Landsat images and conduct spatial analysis. Land cover and land use classification using satellite image analysis includes several steps such as image importing, image gluing, sub setting, sample set preparation, supervised classification etc. While preparing the land use map for any particular year, firstly, the downloaded images of band 2, band 4 and band 7 from different tiles were imported individually. For each of these bands, images of the particular band at different tiles were glued to prepare a mosaic of the tiles. The combined image covers the entire study area. However, it also had portions beyond the study area. Thus, a sub-map was created which contained only the study area. In total three such maps (one for each of the bands 2, 4 and 7) were prepared. These images of band 2, 4 and 7 were used in the later part of land cover mapping. The 7,4,2 band combination was then selected for land cover classification. Four classes of land cover were considered. These are: (1) Agricultural lands, (2) Wetlands, (3) Forestry and (4) Settlements. A sample set was prepared using the selected band combination and a reasonable number of pixels were trained for each of the land cover categories.

The sample set was then used for supervised classification of land cover in the study area. The classification technique used was maximum likelihood classification algorithm. A land cover map of the study area for the particular year was obtained in the process where the study area was classified into the four land cover categories as mentioned earlier. In the classification process, agricultural lands typically included croplands and fallow lands. Forestry primarily included the Sundarbans and other areas with large trees. As the houses and road networks in the rural areas are typically surrounded by large trees, in this study, forestry excluding the Sundarbans was considered as rural settlements and road networks. Settlements primarily included urban settlements, mudlands and barren lands. But the area of rural settlements and road network were also added up to it which has been earlier separated from forestry. This combined area of urban, rural, mudland, barren land and road network has been considered as settlements in this study. Wetlands primarily included shrimp farms, ponds, rivers and the Bay of Bengal. But, rivers and the Bay of Bengal are not among wetlands. Thus, in order to exclude the rivers and the Bay of Bengal, a shape file of rivers in Bangladesh was used. Because the river shape file had some inaccuracies, parts of some rivers were still present in the image alongside the wetlands. These river pixels were also removed. Thus the land cover category 'wetlands' was left only with shrimp farms, ponds. However, in case of post-cyclone images (2008 and 2010), wetlands also included flooded land due to the cyclones, the area of which can be estimated from the difference in wetlands area between the pre- and post-cyclone land cover maps. Table 2 provides the details of Land Cover Classes.

Table 2: Details of Land Cover Classes

Land Cover Classes	Included Land Covers
Agricultural Lands	Croplands and Fallow Lands
Wetlands	Waterbodies excluding Rivers and the Bay of Bengal
Forestry	The Sundarbans
Settlements	Urban and Rural Area, Road Networks, Barren Lands and Mudlands

3.3 Estimation of Agriclutural Lands Affected by Post-Cyclone Flooding

In order to estimate the agricultural lands affected by the flood following the cyclone Sidr, the land cover maps of 2008 and 2007 were analyzed. The wetlands area from the 2008 land cover map in excess to that from the 2007 land cover map was identified and replaced by the land covers of the preceeding year. This resulted in an estimation of agricultural lands and other land cover types being flooded subsequent to cyclone Sidr. The same process was repeated to estimate the agricultural lands being flooded due to cyclone Aila, by comparing the land cover maps of 2010 and 2009.

3.4 Identification of Production Loss due to Post-Cyclone Flooding

Two different rule based techniques has been developed to identify crops and fisheries productions affected by each event of flooding followed after the cyclone Sidr and cyclone Aila. To identify production loss due to post-Sidr flooding, the production of different crops and fisheries in 2006-07 (P07), 2007-08 (P08) and 2008-09 (P09) were analyzed. If the temporal variation of production followed the pattern: P07>P08<P09, then the production was considered to be affected by the subsequent flooding. Again, in case of post-Aila flooding, the production of different crops and fisheries in 2007-08 (P08), 2008-09 (P09) and 2009-10 (P10) were analyzed. If the temporal variation of production followed the pattern: P08<P09>P10, then the production was considered to be affected by the long-term flooding due to cyclone Aila.

4. RESULTS AND DISCUSSION

The main focus of the study is to assess the long-term flooding due to the two major cyclonic events Sidr and Aila and its impacts on certain ecosystem services such as agricultural lands, crop production and fisheries production. Therefore, the temporal variations of all land cover classes have been quantified and presented in Figure 1, but only the wetlands and agricultural land cover changes are shown in Figure 2. Discussion over forestry (The Sundarbans) is omitted as its area has been found to be almost constant throughout the period.

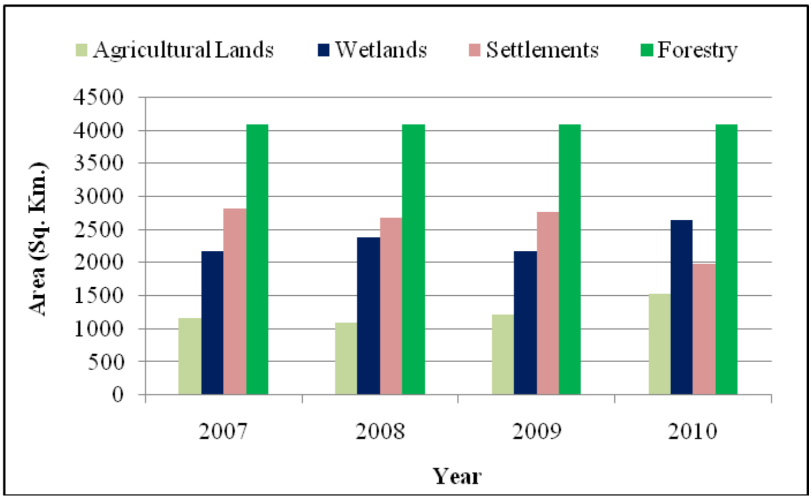

Figure 1: Temporal variations of different land cover classes in the study area

It has been assessed that in January 2008 the wetlands area in the study area increased by 11% from that in January 2007. However, in the following year it decreased by 12% but in January 2010, it again increased by 25%. Such sudden and sharp changes in wetlands area can be largely attributed to the widespread inland flooding caused by the cyclonic storm surges in 2007 and 2009 and the excess waters can be recognized as flooded areas. It has been also found that the study area consists of more wetlands than agricultural lands and also that a lagre portion of it is settlements. Thus, most of the flooded areas has been identified to be settlements and wetlands specially shrimp firms. However, significant agricultural land area is also found to be affected by the floodings after cyclone Sidr and cyclone Aila. As a result, production of several crops in the study area decreased significantly in the subsequent year of each of the cyclonic events. Table 3 shows the estimated agricultural lands flooded after the cyclones and Table 4 presents crop productions lost due to post-cyclone floodings. The post-Aila flooding also affected the production of shrimps significantly. However, it has been found that cyclone induced floods rarely affected the fisheries production. The only exception has been the fisheries production of Baors. Table 5 furnishes fisheries production affected by cyclone Sidr & cyclone Aila.

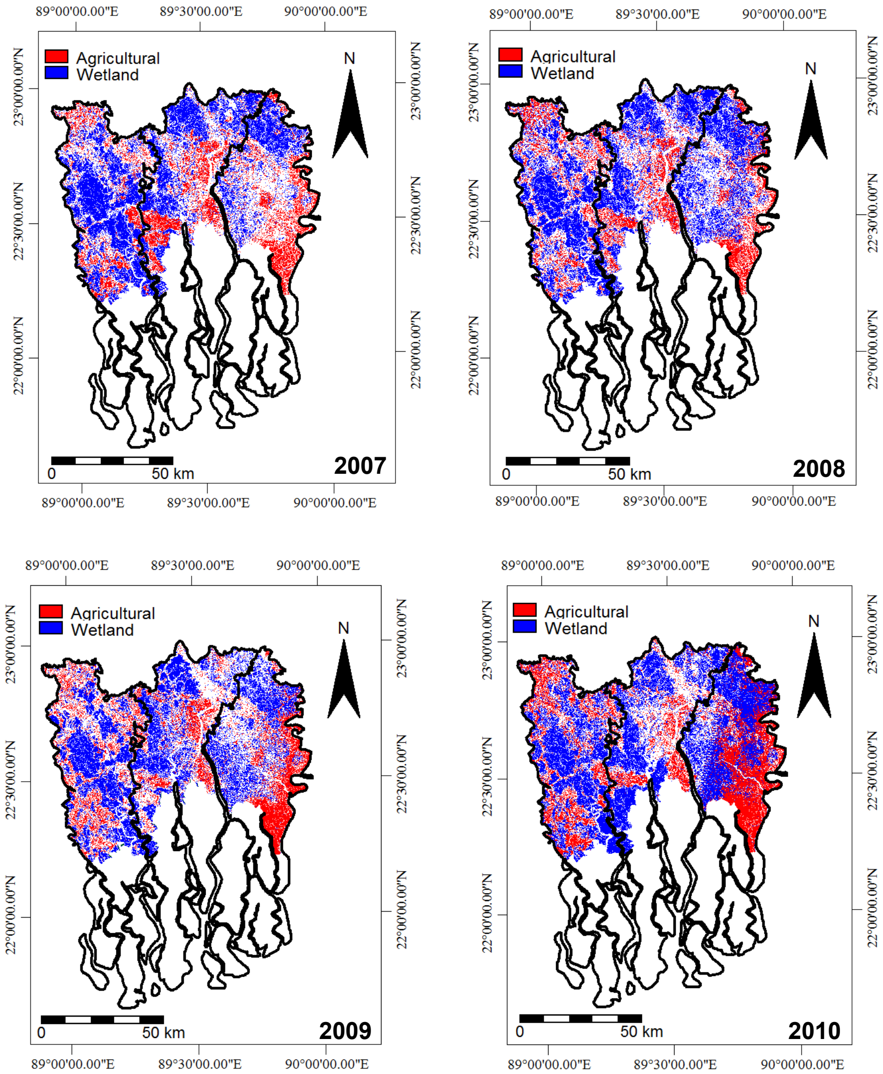

Figure 2: Spatial coverage of agricultural lands and wetlands in the study area at different years

Table 3: Estimated Agricultural Lands Flooded after the Cyclones

	Area (sq. km.)	% Agricultural Land
Post-Sidr (2008)	30	2.7
Post-Aila (2010)	110	6.7

Table 4: Crop Production Affected by Cyclone Sidr & Cyclone Aila

Production Decrease (%)			
Post-Sidr (2008)		**Post-Aila (2010)**	
Aus	8.6	Aus	10.9
Aman	14.9	Aman	11.8
Rice (Total)*	1.8	Rice (Total)*	1.2
Rabi Brinjal	3.7	Rabi Brinjal	19.7
		Wheat	16.1
		Jute	22.6
		Tomato	3.5

*Rice (Total) = Aus+Aman+Boro

Table 5: Fisheries Production Affected by Cyclone Sidr & Cyclone Aila

Production Decrease (%)			
Post-Sidr (2008)		Post-Aila (2010)	
Baor	15.4	Baor	37.0
		Shrimp	9.2

5. CONCLUSIONS

Results of this study reveal that both cyclone Sidr and cyclone Aila significantly affected the production of several ecosystem services in the southwest coastal zone of Bangladesh. In most cases the impact of cyclone Aila has been more severe than that of cyclone Sidr. Interestingly, the top wind speed of cyclone Sidr was more than twice than that of cyclone Aila. But Aila impacted during the high tide while Sidr made landfall during low tide. This indicates that the tide situation during a cyclonic event governs the damage intensity by the cyclone more than its top wind speed. Among the major varieties of rice, production of Aus and Aman rice decreased by 10.9 and 11.8 percent respectively from 2009 to 2010. However, production of Boro rice increased by 13.1% during this period. Such contrast can be attributed to technological advancements and better management practices which resulted in much increased production in spite of decreased command area. Another reason could be that, the cultivation area of such crops was not flooded or damaged by the storm surges. Similarly, significant decrease in shrimp production from 2009 to 2010 has been found which can be attributed to the shrimp firms being washed out by the storm surge induced floodings. Production data of more years can be analyzed to estimate production loss more accurately. Further research should be conducted to identify factors other than cyclone flooding that may have caused the sudden increase in wetlands so that the impact of cyclones can be estimated more precisely. Also, cyclone impacts can be attempted to correlate with storm properties, which may help to estimate production loss due to cyclones in advance. Proper structural and non-structural measures should be taken to reduce the impacts of cyclonic storms in the country.

REFERENCES

BBS (2012). “Bangladesh Household and Population Census 2011 – Community Report”. Bangladesh Bureau of Statistics. Statistics and Informatics Division, Ministry of Planning, Government of Bangladesh.

Boyd, J., and Banzhaf, S. (2007). “What are ecosystem services? The need for standardized environmental accounting units.” Ecological Economics, Vol. 63(2), 616-626.

CIA (2013). “The World Factbook: Bangladesh.” Central Intelligence Agency, United States of America, https://www.cia.gov/library/publications/the-world-factbook/geos/bg.html.

DMB (2009). “Summary of Cyclonic Storm AILA.” Disaster Management Bureau, 10 June 2009, Dhaka, Bangladesh.

ESPA-Deltas (2013). Assessing Health, Livelihoods, Ecosystem Services and Poverty Alleviation in Populous Deltas, http://www.espadelta.net/.

ILWIS (2012). Integrated Land and Water Information System, http://52north.org/.

IWM (2009). “Preparation of Inundation Map Considering the Decay Factors for Landuse, Geomorphology and Slope.” Report submitted to Comprehensive Disaster Management Programme, Ministry of Food and Disaster Management, Government of the People’s Republic of Bangladesh, Dhaka.

Jackobsen, F., Azam, M.H., Ahmed, M.M.Z. and Kabir, M.M. (2006). “Cyclone Storm Surge Levels along the Bangladeshi Coastline in 1876 and 1960-2000.” Coastal Engineering Journal, Volume 48, No. 3.

Miyan, M.A. (2009). “Monsoon in Bangladesh.” Proceedings of the Changes and Adaptation: Asian Monsoon Years AMY-6, Kunming, China, November 30-December 01, 2009.

PDO-ICZMP (2003). “A System Analysis of Shrimp Production (WP 014).” Program Development Office for Integrated Coastal Zone Management Plan, Water Resources Planning Organization, Government of Bangladesh.

SMRC (1998). “The Impact of Tropical of Cyclone on the Coastal Regions of SAARC Countries and Their Influence in the Region.” SAARC Meteorological Research Center Report No. 1.

UNDP (2004). “Reducing Disaster Risk: A Challenge for Development – A Global Report.” United Nations Development Programme, Bureau for Crisis Prevention and Recovery, New York.

STUDY ON THE PERFORMANCE OF DIFFERENT COAGULANTS FOR TURBIDITY AND COLOR REMOVAL

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ABSTRACT

Water is a prime necessity and potable water is now a basic need of time for human being. The two third of the world contain water; out of this amount, 97.5% is unusable due to presence of salt. To solve this problem, salt free surface water can be an alternative source for drinking water. However, surface water is usually rich in turbidity and color. Turbidity imparts a great problem in water treatment. The current study aims at presenting the objective approach of jar test for the purpose of finding out optimum dose for different coagulants (including chemical and bio-coagulant) for both turbidity and color removal. Based on that optimum dose, the removal performance of turbidity and color was also investigated. The effect of pH, alkalinity and temperature were studied and optimized. The achieved optimum dose was used to compare the removal performance of turbidity and color at different slow mixing period. Among four different coagulants, namely aluminum sulphate, ferrous sulphate, ferric chloride and PG-M, the lowest optimum dose was found for ferric chloride as 28.0 mg/L for turbidity and 24.0 mg/L for color removal. The highest removal performance was obtained for a slow mixing period of 20.0 to 25.0 minutes.

Keywords: *Coagulants, Optimization, Removal Performance, Turbidity, Color.*

1. INTRODUCTION

Coagulants are either chemical salt or polymers (bio-coagulant) which works on the principles of neutralizing the electrical charge to clump together. Major contaminants generally found in surface water supplies includes: clays, micro-organisms, natural organic matter (NOM), algae and trace metals etc. which are responsible for producing color and turbidity in water. Color caused by dissolved and colloidal form of impurities is called true color and that caused by suspended matter in addition to dissolved and colloidal matters, is called apparent color. Ground water may show color due to the presence of iron compound. Color intensity increases with the increases of pH. The term turbid is applied to water containing suspended matter that interferes with passage of light through the water i.e. where visual depth is restricted. The agglomeration (coagulation-flocculation) process in water treatment is the most important in contaminant i.e. turbidity and color removal. This process which is induced by the addition and dispersion of chemicals destabilize particles in solution by neutralizing their charge [Drikas et.el. 2001]. Solid-liquid separation processes of coagulation and filtration, when optimized can remove all organic, inorganic and suspended matter to a level below water quality standards in most cases. Most commonly used coagulants in water treatment process are aluminum or iron salts. Natural organic matter is by far the largest sources of organic material in raw water and it comprises both particulate and dissolved components [Exall et. el. 2000]. Iron salts are more effective for intermediate molar mass organics. Both aluminum and iron salts are poorly effective for organic of low molar mass [Matilainen et.el. 2005]. The mechanism by which most metal salts coagulate natural organic matter include: (1) formation of insoluble complexes at low pH level through neutralization of negatively charged NOM by positively charged metal hydrolysis products, or (2) the adsorption of NOM into the metal hydrolysis precipitate by high dose of metal coagulant at high pH level [Bell Ajy et.el., 2000]. Low turbid water is hard to coagulate due to low concentrations of stable particles. Coagulation is a vital process in the treatment of both surface water and industrial wastewater. It is important for water supply engineers as turbid and colored water is not aesthetically acceptable to people. Its application includes removal of dissolved and suspended chemical and organic species. In water treatment process besides removing turbidity and color it also plays an important role in disinfection process. Disinfection by chlorination of waters containing natural organics (which produce color) results in the formation of chloroform, other trihalomethanes and a range of other chlorinated organic leading to problems. Disinfection with high turbid water, pathogenic organisms may inter into the particles and protect from the disinfectant. Also for filtration turbid water is not suitable as it cause quick clogging of filter bed which necessitates the use of pretreatment plant [Sawyer et. al. 1994]. Coagulation helps to avoid all these above problems and makes water as a potable one. Turbidity and color measurements imply particular importance in the field of water supply. They have limited use in the field of domestic and industrial waste treatment. Water supply obtained from rivers usually requires chemicals flocculation because of high turbidity and color. Turbidity and color measurement using coagulation are used to determine the effectiveness of the treatment produced with different coagulants and the doses needed. Thus they aid in selection of the most effective and economical coagulant to use. Such information is necessary to design facilities for feeding the coagulants and for their storage. Besides water supply many industrial processes also requires the use of color and turbid free water. Before a chemical treatment plant is designed, research should be conducted to ascertain the best chemicals to use and amounts required [Sawyer et. al. 1994].

2. METHODOLOGY

2.1 Source of Water

The natural source of water was obtained from a pond in the KUET campus. This water was chosen based on its low turbidity (about 10.0 NTU) and relatively high amount of color (125.0 to 172.0 Pt-Co units). Water was stored in the laboratory using large type of bottle and the experiments were performed as soon as after collecting the source water.

2.2 Instrumentation

Used instruments are: Coagulation (stirring) device, Turbidity meter, Spectrophotometer, pH meter, Burette, Pipette, Glass beakers (600 ml, 6 Nos.), Sample cell, Balance, Marker, Tape, Tissue paper etc.

2.3 Chemicals Used

For this study among various chemicals and bio- coagulants the following four types coagulant were selected:

a) Alum coagulant$\{Al_2(SO_4)_3.14H_2O\}$
b) Ferric chloride coagulant ($Fecl_3.6H_2O$)
c) Ferrous sulphate coagulant ($FeSO_4.7H_2O$)
d) PG-M coagulant (hybrid type coagulant).

2.4 Test Procedure

KUET pond water sample was treated with different concentrations of the four selected coagulants. The water was low in turbid but comparatively high concentrations of color. Initial turbidity was around 10.0 NTU and initial color was around 172.0 Pt-Co during the overall experiments. Therefore, the variation of initial turbidity and color was not so significant amount that can affect the optimum dose. The effect of pH, alkalinity and temperature were examined. A series of jar tests were carried out to determine the optimum dose and removal performance of turbidity and color. All the tests were done at 30°C temperature. Figure1 shows the experimental setup of the jar test.

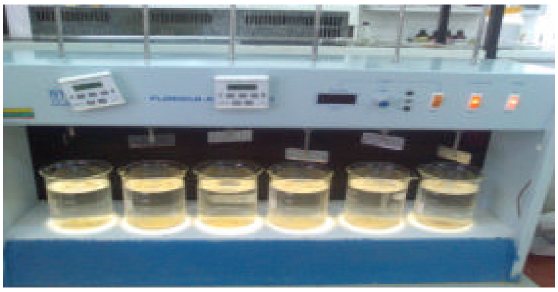

Figure 1: Experimental setup of a jar test.

3. RESULTS AND DISCUSSIONS

3.1 Determination of Optimum Doses of Coagulants

3.1.1 PG-M optimization

Table 1 shows remaining turbidity and color with varying dose for PG-M coagulant. Figure 2 shows required optimum dose for PG-M coagulant in terms of turbidity removal is 60.0 mg/L. It was the highest dose requirement among the selected coagulants in terms of both turbidity and color removal in this study.

Table 1: Remaining turbidity and color with varying dose of PG-M coagulant

PG-M dose (mg/L)	Turbidity remaining (NTU)	Color remaining (Pt-Co)
20.0	10.0	78.0
32.0	6.67	27.0
42.0	4.74	14.0
54.0	6.12	35.0
70.0	6.62	54.0
90.0	6.85	65.0

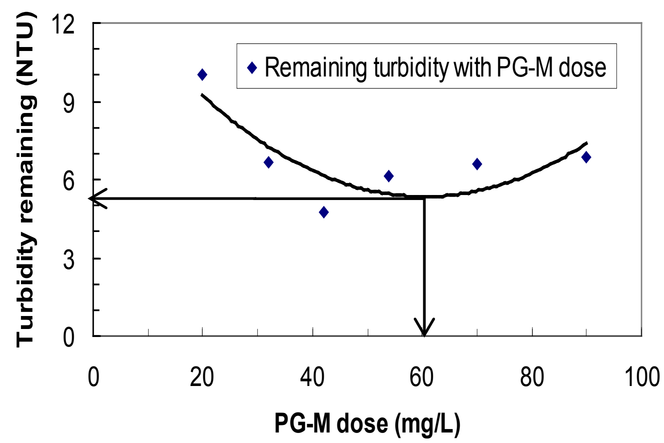

Figure 2: Determination of optimum dose in terms of turbidity removal

Figure 3 shows optimum dose for PG-M coagulant in terms of color removal is 52.0 mg/L.

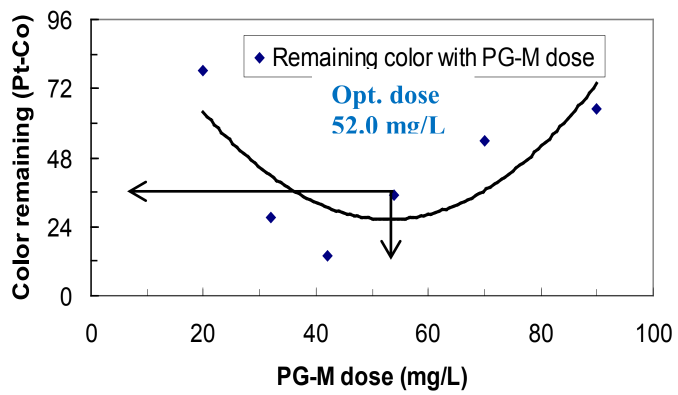

Figure 3: Determination of optimum dose in terms of color removal.

3.1.2 *Alum optimization*

Table 2 shows remaining turbidity and color with varying dose for alum coagulant. Figure 4 shows optimum dose for alum in terms of turbidity removal is 34.0 mg/L and Figure 5 shows optimum dose for alum in terms of color removal is 30.0 mg/L.

Table 2: Remaining turbidity and color with varying dose of alum

Alum dose (mg/L)	Turbidity remaining (NTU)	Color remaining (Pt-Co)
10.0	5.54	47.0
20.0	2.35	11.0
30.0	1.67	4.0
40.0	1.49	28.0
50.0	3.64	35.0
60.0	5.18	73.0

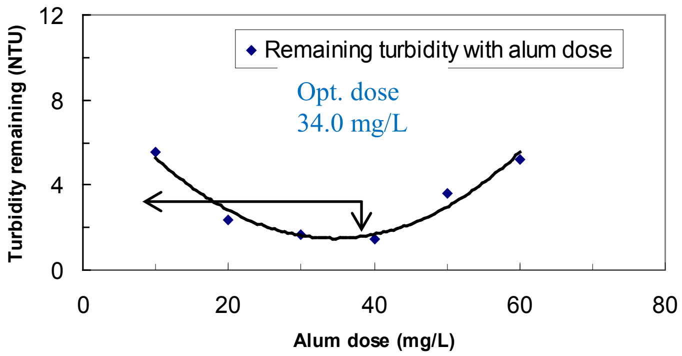

Figure 4: Determination of optimum dose in terms of turbidity removal

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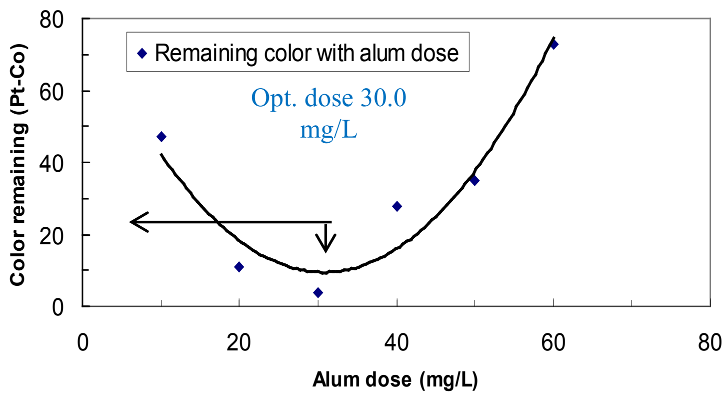

Figure 5: Determination of optimum dose in terms of color removal.

3.1.3 *Ferric chloride optimization*

Table 3 shows remaining turbidity and color with varying dose for ferric chloride coagulant. Figure 6 shows optimum dose for $Fecl_3$ in terms of turbidity removal is 28.0 mg/L. This was the lowest dose requirement in terms of turbidity removal (Table 5) and Figure 7 shows optimum dose for $Fecl_3$ in terms of color removal is 24.0 mg/L. Among four different coagulants it was the lowest dose in terms of both turbidity and color removal.

Table 3: Remaining turbidity and color with varying dose of ferric chloride

Ferric chloride dose (mg/L)	Turbidity remaining (NTU)	Color remaining (Pt-Co)
10.0	4.41	46.0
20.0	2.85	14.0
30.0	1.65	27.0
40.0	3.10	54.0
45.0	4.16	63.0
50.0	4.94	71.0

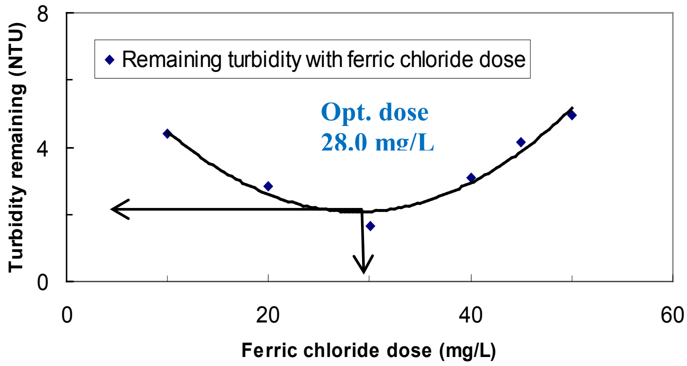

Figure 6: Determination of optimum dose in terms of turbidity removal.

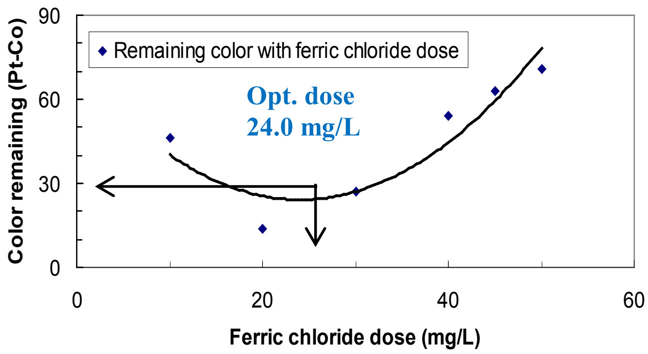

Figure 7: Determination of optimum dose in terms of color removal.

3.1.4 Ferrous sulphate optimization

Table 4 shows remaining turbidity and color with varying dose of ferrous sulphate.

Table 4: Remaining turbidity and color with varying dose ferrous sulphate

Ferrous sulphate dose (mg/L)	Turbidity remaining (NTU)	Color remaining (Pt-Co)
10.0	10.0	158.0
20.0	8.78	121.0
30.0	6.61	8.0
40.0	5.78	76.0
50.0	3.65	81.0
60.0	5.94	87.0

Figure 8 shows optimum dose for ferrous sulphate ($FeSO_4$) in terms of turbidity removal is 56.0 mg/L and Figure 9 shows optimum dose for $FeSO_4$ in terms of color removal is 40.0 mg/L.

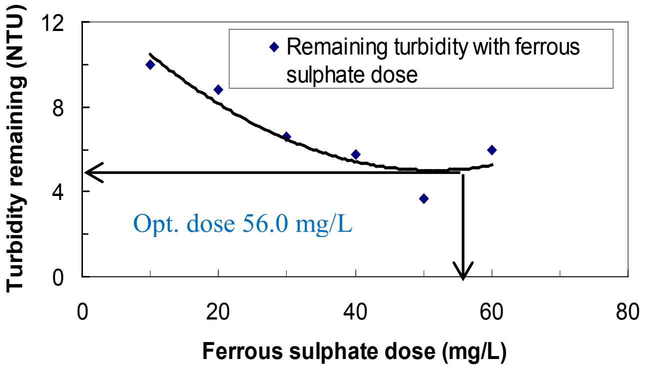

Figure 8: Determination of optimum dose in terms of turbidity removal

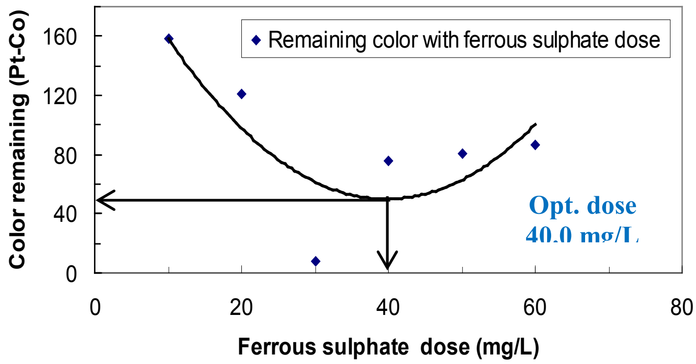

Figure 9: Determination of optimum dose in terms of color removal

3.1.5 *Summary of optimum doses of coagulants*

The optimum value of alkalinity and pH were 75.0 mg/L and 6.90 respectively. Optimum dose of each aforementioned selected coagulant for the pond water was determined by trial method e.g. PG-M ranging between 5.0 to 100.0 mg/L and best fitted graph was found within 20.0 to 90.0 mg/L (Figure 2 and Figure 3) by trial in terms of both turbidity and color removal. For alum and ferrous sulphate best fitted graph was found within 10.0 to 60.0 mg/L (Figure 4, Figure 5, Figure 8 and Figure 9) whereas for ferric chloride was found within 10.0 to 50.0 mg/L (Figure 6 and Figure 7). Optimum dose was found out in case of both turbidity and color removal of each coagulant. The obtained optimum dose is tabulated in Table 5.

Table 5: Optimum dose of different coagulants

Coagulant type	Optimum dose (mg/L)			
	Alum	Ferric chloride	PG-M	Ferrous sulphate
In terms of turbidity removal	34.0	28.0	60.0	56.0
In terms of color removal	30.0	24.0	52.0	40.0

3.2 Turbidity and Color Removal Performance

Turbidity and color removal performance at the obtained optimum dose was studied with respect to slow mixing period. Experiments were performed at optimum dose with varying slow mixing period from 15.0 to 40.0 minutes.

3.2.1 Optimum dose for turbidity removal

Table 6 and Figure 10 show removal performance was best for alum coagulant than others and ferrous sulphate shows poor removal performance in terms of turbidity removal at optimum dose for turbidity removal. Removal performance was optimized near 25.0 minutes slow mixing period considering each coagulant.

Table 6: Remaining turbidity with slow mixing period

Slow mixing period (minutes)	Remaining turbidity (NTU) at optimum dose for turbidity removal			
	Ferric chloride	Alum	PG-M	Ferrous sulphate
15.0	2.27	1.62	3.53	7.57
20.0	2.9	1.46	3.02	5.67
25.0	0.97	0.91	2.38	3.52
35.0	1.64	1.13	1.94	3.79
40.0	1.4	3.11	1.79	6.39

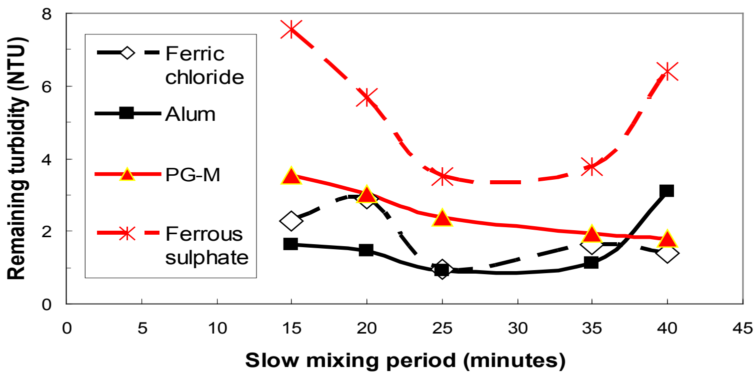

Figure10: Turbidity removal efficiency of selected coagulants at optimum dose for turbidity removal

Table 7: Remaining color (Pt-Co) with slow mixing Period at optimum dose for turbidity removal

Slow mixing period (minutes)	Remaining color (Pt-Co) at optimum dose for turbidity removal			
	Ferric chloride	Alum	PG-M	Ferrous sulphate
15.0	74.0	42.0	62.0	85.0
20.0	26.0	29.0	21.0	55.0
25.0	8.0	11.0	27.0	25.0
35.0	36.0	19.0	62.0	41.0
40.0	32.0	14.0	29.0	53.0

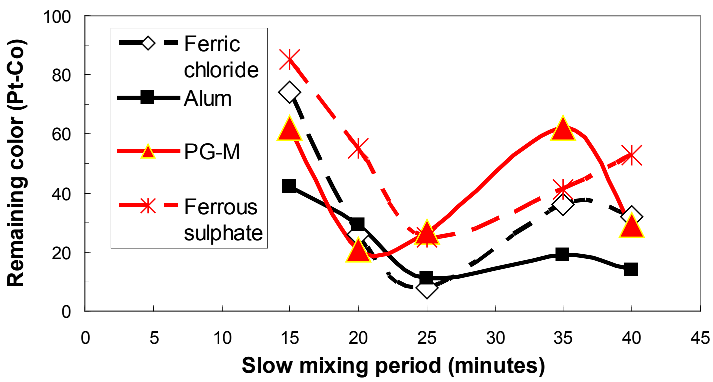

Figure11: Color removal efficiency of selected coagulants at optimum dose for turbidity removal

Table 7 and Figure 11 show removal performance was better for alum coagulant than others and ferrous sulphate shows relatively better removal performance in terms of color removal at optimum dose for turbidity removal. Here also removal performance was optimized near 25.0 minutes slow mixing period considering each coagulant. After removing relatively high color, remaining color for alum and ferric chloride was within 15.0 Pt-Co (Bangladesh standard) at optimized slow mixing period.

3.2.2 Optimum dose for color removal

Table 8 and Figure12 show removal performance for alum, PG-M and ferric chloride was more or less same in terms of turbidity removal at optimum dose for color removal and was optimized near slow mixing period 20.0 minutes.

Table 8: Remaining turbidity (NTU) with slow mixing period at optimum dose for color removal

Slow mixing period (minutes)	Remaining turbidity (NTU) at optimum dose for color removal			
	Ferric chloride	Alum	PG-M	Ferrous sulphate
15.0	3.71	1.55	2.2	5.43
20.0	1.27	1.04	2.34	5.12
25.0	1.91	1.93	1.92	4.74
35.0	1.97	1.65	2.0	5.96
40.0	1.34	2.27	3.17	6.44

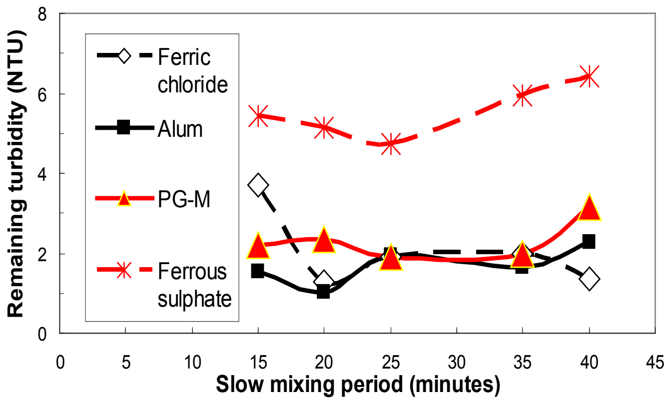

Figure12: Turbidity removal efficiency of selected coagulants at optimum dose for color removal

Table 9: Remaining color (Pt-Co) with slow mixing Period at optimum dose for color removal

Slow mixing period (minutes)	Remaining color (Pt-Co) at optimum dose for color removal			
	Ferric chloride	Alum	PG-M	Ferrous sulphate
15.0	37.0	23.0	30.0	57.0
20.0	14.0	18.0	13.0	52.0
25.0	34.0	29.0	10.0	41.0
35.0	10.0	19.0	29.0	72.0
40.0	12.0	11.0	2.0	63.0

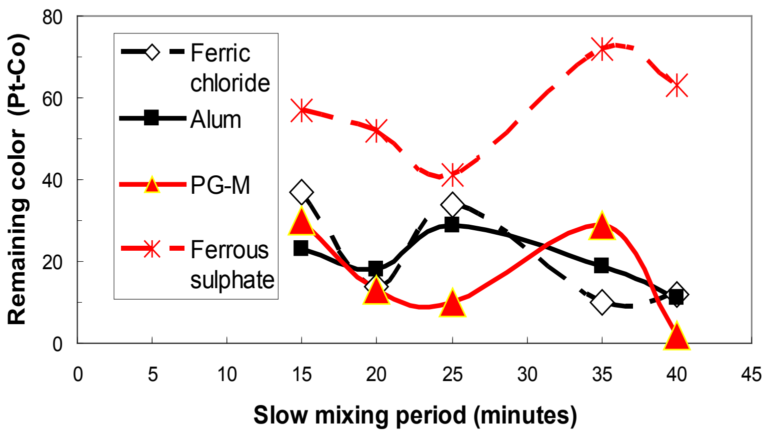

Figure13: Color removal efficiency of selected coagulants at optimum dose for color removal

Table 9 and Figure13 show removal performance was relatively better for PG-M than other coagulants in terms of color removal at optimum dose for color removal. Here also ferrous sulphate shows poor removal performance than other coagulants. Removal performance was optimized at slow mixing period 20.0 minutes for alum, PG-M and ferric chloride and for ferrous sulphate optimized at slow mixing period 25.0 minutes.

4. CONCLUSIONS

From the overall study the following conclusions can be drawn on the basis of results obtained:

a) In this study optimum dose for ferric chloride coagulant was found the lowest among all other coagulants in terms of both turbidity and color removal.
b) In case of PG-M the highest optimum dose requirement was found.
c) Alum coagulant has shown best removal performance in terms of optimum dose of turbidity removal.
d) Among four selected coagulant ferrous sulphate has shown poor removal performance than other three coagulants.
e) PG-M has shown best removal performance in terms of optimum dose of color removal.
f) From overall consideration removal performance was optimized around slow mixing period 20.0 to 25.0 minutes, considering both turbidity and color removal.
g) PG-M produces comparatively less sludge and may be safe from side effect of metal residual that is generally formed from chemical coagulants.

NOMENCLATURE

NOM : Natural organic matter
NTU : Nephelometric turbidity unit
Pt-Co : Platinum-cobalt unit, for color measurement
KUET : Khulna University of Engineering & Technology, Khulna-9203, Bangladesh
PG-M : Hybrid type coagulant

REFERENCES

Bell-Ajy K., Abbasadegan M., Ibrahim E., Verges D. and LeChevallier M., (2000). Conventional and Optimized Coagulation for NOM Removal, J. AWWA, 29(10) pp.44-58.

Drikas M., Chow C. W. K., House J., and Burch M. D., (2001). Using Coagulation, Flocculation and Settling to Remove Toxic Cyano-Bacteria, J. AWWA., pp. 100-110.

Exall K. N., and Vanloon G. W., Using Coagulants to Remove Organic Matter, J. AWWA, pp.93-102, (2000).

Matilainen A., Lindqvist N. Tuhkanen T., (2005). Comparison of the Efficiency of Aluminum and Ferric Sulfate in the Removal of Natural Organic Matter during Drinking Water Treatment Process, Environmental Technology, 26(8), 867-876.

Sawyer N. Clair, McCarty L. Perry, Parkin F. Gene, (1994). Chemistry for Environmental Engineering, 4^{th} edition, McGraw-Hill, Inc., pp. 440-446.

EVALUATION OF CR(VI) REMOVAL EFFICIENCY FROM WASTEWATER USING BIO-CHAR

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ABSTRACT

In recent years, chromium contamination has become one of the significant environmental problems. Different industries such as electroplating, tannery, automobile, pigments, paper and pulp, fertilizer, textile, steel, and metal finishing are the main contributor to chromium poisoning. Hexavalent chromium has severe adverse impacts because of its oxidizing potential and easy permeation through biological membranes. The health problems associated with Cr(VI) include gastrointestinal, dermatological, and respiratory problems, stomachs dysfunction, ulcers, immune systems weakness, kidney and liver damage, alteration of genetic material, lung cancer, asthmatic bronchitis, bronchospasms, edema etc. The Cr(VI) concentration in wastewater should not exceed 0.05 mg/L according to the international environmental standards. There is currently no known treatment method to eradicate Cr(VI) poisoning effectively. Hence, the aim of the present study is to evaluate the removal efficiency of Cr(VI) from wastewater using bio-char as a low cost adsorbent. Bio-char is a black granular solid material prepared by pyrolysis of wood chips at 700 ^{0}C. The particle size of bio-char used was in between 53 and 250 µm. Batch sorption experiments were conducted by using a synthetic wastewater. The experimental parameters such as pH, contact time, initial Cr(VI) concentration, and bio-char dosages were optimized for the maximum removal of Cr(VI) during the batch experiments. The concentrations of Cr(VI) were measured by using a graphite furnace equipped atomic absorption spectrophotometer (GF-AAS) with an air-acetylene flame and chromium hollow cathode lamp at a slit of 2 nm and wavelength of 357.9 nm. The results illustrate that the Cr(VI) removal efficiency was up to 100 % at a pH of 2 with initial Cr(VI) concentration of 10 mg/L using 4 g of bio-char dosages in 200 mL of solution volume after 5 hours of reaction time. The maximum sorption capacity of bio-char was 1.717 mg/g for initial Cr(VI) concentration of 500 mg/L after 5 hours of sorption reaction. Consequently, it can be concluded that the bio-char is an effective sorbent for removing Cr(VI) from the wastewater.

Keywords: *Contamination; Hexavalent chromium; Bio-char; Batch test; Sorption*.

1. INTRODUCTION

Chromium contamination has been found in many parts of the world including developed and developing countries where its industrial application is common. The worst situation for chromium poisoning has been noted in areas where industries such as electroplating, tannery, automobile, pigments, paper and pulp, fertilizer, textile, steel, and metal finishing are frequently located. Chromium containing industrial effluents eventually contaminate nearby soils and surface waters. Soil also releases chromium into streams, lakes, rivers, and groundwater. In particular, groundwater contaminated with this chromium is highly hazardous, especially in areas where groundwater is extracted for drinking water.

The major oxidation states of chromium include trivalent and hexavalent chromium which have been detected in industrial effluents. In groundwater around chromium ore processing facilities, Cr(VI) concentration of up to 91 mg/L has been reported [1]. Concentration of Cr(VI) in the effluent of electroplating industry has been found to range from 3 to 50 mg/L [2]. The concentration of chromium in some municipal and industrial wastes was reported to be as high as 1993 mg/kg [1]. International environmental standards recommend that chromium concentration in wastewater should not exceed 5 mg/L for Cr(III) and 0.05 mg/L for Cr(VI) [3]. The allowable limit of chromium concentration for humans is 0.05 µg/L for drinking water, and 0.5 – 500 µg/m^3 for atmospheric exposure [4].

Hexavalent chromium has severe adverse impacts because of its oxidizing potential and easy permeation through biological membranes. Water soluble Cr(VI) is easily absorbed by human tissues, and crosses body's cell membranes rapidly [5]. The health problems associated with Cr(VI) include gastrointestinal, dermatological, and respiratory problems, stomachs dysfunction, ulcers, immune systems weakness, kidney and liver damage, alteration of genetic material, lung cancer, asthmatic bronchitis, bronchospasms, edema etc. [4, 5, 6].

In view of the above circumstances, suitable treatment methods for hexavalent chromium removal from industrial wastewater should be developed and applied. The common methods of treatment include *precipitation* [7, 8, 9, 10], *ion exchange* [11, 12, 13, 14, 15], *membrane separation* (i.e. ultrafiltration and reverse osmosis) [16, 17, 18, 19], *coagulation and electrocoagulation* [20, 21, 22]. However, most of the above mentioned treatment technologies have limitations which include one or more: generation of toxic sludge, high operational and maintenance expenditure, high energy requirements, too long treatment time, and poor removal efficiency [9, 22].

In addition, adsorption process has been received significant attention for the removal of metal ions [23, 24, 25]. The significant benefits of adsorption process include effective and economical contaminant removal, recovery and recycling of adsorbed metals from adsorbent, less amount of sludge production, simple process and high removal efficiency [9]. As an effective and economical adsorbent, activated carbon has been widely implementing for wastewater treatment for the past few decades [9, 23, 24, 25]. In addition, some other low cost adsorbents have been reported for chromium removal from wastewater include *maple saw dust* [26], *bentonite and expanded perlite* [27], *chitin and chitosan* [28], *coal* [29], *nano-particles* [30].

However, as a low cost adsorbent, bio-char has been rarely investigated [10, 31]. Thus, a significant knowledge gap exists for the possible use of this low-cost material as economically viable adsorbents. In this present study, bio-char generated from wood chips was used as adsorbent to evaluate its performances for Cr(VI) removal from waste-water.

2. METHODOLOGY

2.1 Bio-char

The bio-char used in this study was purchased from Phoenix Energy, San Francisco, California, USA. It is a black granular solid material prepared by pyrolysis of wood chips at 700 ^{0}C. The purchased bio-char has a particle size between 53 and 250 µm.

2.2 Sorption procedure

In this study, a batch sorption process was employed to find out the optimum pH, bio-char dosage, and initial Cr(VI) concentration.

The stock solution of 1000 mg/L of Cr(VI) was prepared by dissolving 2.8269 g of potassium dichromate ($K_2Cr_2O_7$) salt (Sigma-Aldrich LTD, USA, product code no 101064403) in deionized water. The stock solution of Cr(VI) was subsequently diluted using deionized water to the required working concentration prior to use in the batch tests. The pH variation was conducted by either 15.8M HNO_3 or 1M NaOH. The Cr(VI) and bio-char mixture was placed on the shaker at 150 rpm at room temperature.

About 4 mL samples were collected using separate syringe at 0, 0.5, 1, 2, 3, 4, 5, 6, 7, 8, and 24 hours and 4 mL/day from day 2 to day 6. Collected samples were then filtered through 0.45 µm syringe filter. Filtered samples were stored in glass bottle and preserved in the refrigerator at 4°C with sample pH below 2 to avoid the degradation of Cr(VI) concentration.

Samples were then analyzed to measure the Cr(VI) concentrations using a graphite furnace equipped atomic absorption spectrophotometer (GF-AAS) with an air-acetylene flame and chromium hollow cathode lamp at a slit of 2 nm and wavelength of 357.9 nm. Prior to analyze the sample, the GF-AAS instrument was calibrated regularly.

2.2.1 Evaluation of optimum parameters (pH, bio-char dosage, and initial Cr(VI) concentration) and equilibrium studies

The optimum pH was determined by changing solution pH of 1, 2, 3, 5, 7, and 9 with bio-char dosage of 2 g/200 mL solution. Moreover, different bio-char dosages of 1 g, 2 g, 3 g, 4 g, and 5 g in 200 mL solution were evaluated to know the optimum dosage at pH of 2. The initial Cr(VI) concentration was kept at 10 mg/L to investigate optimum pH and bio-char dosage. The initial Cr(VI) concentrations of 10, 50, 100, 300, and 500 mg/L were evaluated with 2 g of bio-char in 200 mL solution at pH 2 in order to obtain its optimum value.

The Cr(VI) concentrations at equilibrium (C_e) and at any particular time (C_t) were evaluated for different initial Cr(VI) concentrations (C_0) of 10, 50, 100, 300, and 500 mg/L maintaining the pH and bio-char dosage constant at the optimum value. These concentrations (C_0, C_e, C_t) were applied to calculate the sorption capacity of bio-char at a particular time (q_t) and at equilibrium (q_e) according to the mass balance Eq. (1) and Eq. (2), respectively [9]. Afterward, these sorption capacities (q_e, q_t) were used to draw the sorption isotherms and kinetics model plots.

$$q_t = \left(\frac{C_0 - C_t}{m}\right)V \qquad (1)$$

$$q_e = \left(\frac{C_0 - C_e}{m}\right)V \qquad (2)$$

where C_0 is the initial Cr(VI) concentration in mg/L, C_t is the Cr(VI) concentration at a particular time in solution in mg/L, C_e is the equilibrium Cr(VI) concentration in mg/L, V is the volume of the solution in litter (L), and m is the dry mass of bio-char in gram (g).

3. ILLUSTRATIONS

3.1 Evaluation of the optimum sorption parameters

3.1.1 Effects of pH

The Cr(VI) removal efficiencies decreased consistently with increased pH from 1 to 9 except pH 2 and were slightly increased with time for each pH (Fig. 1). At pH 2, the Cr(VI) removal efficiency was around 80 % after 30 minutes of sorption reaction and increased to 98 % after 24 hours which was the highest Cr(VI) removal efficiency as compared to the efficiencies found at other pH values. The removal efficiencies are found below 30 % for pH 1, 3, 5, 7, and 9. Therefore, the optimum pH was taken as 2 for the highest removal of Cr(VI) from solution. Other studies also recommended similar optimum pH values [23, 24, 25, 31].

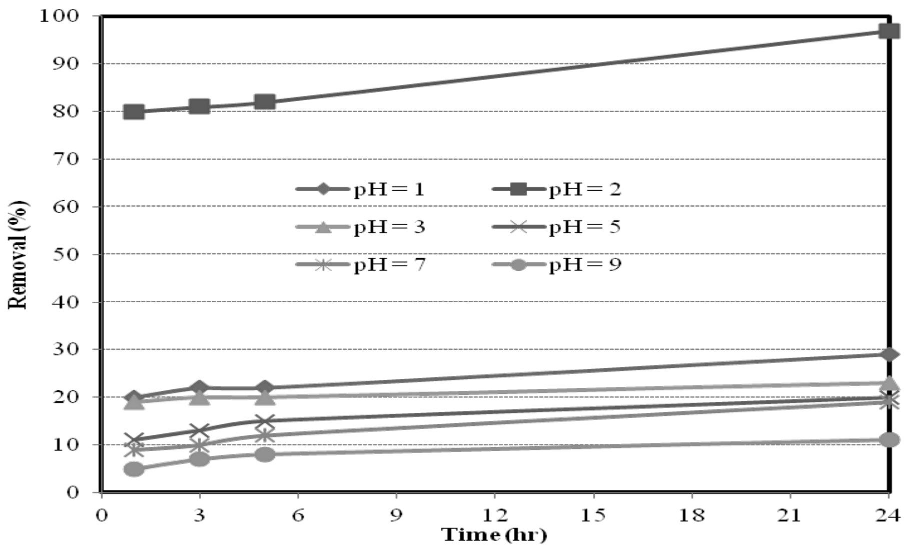


Figure 1: Removal efficiencies of Cr(VI) at various times with different pH values using bio-char as sorbent. Solution volume: 200 mL; initial Cr(VI) concentration: 10 mg/L; amount of bio-char: 2 g.

3.1.2 Effects of bio-char dosages

The removal efficiencies of Cr(VI) increase rapidly with increased bio-char dosage from 1 to 5 g (Fig. 2). The highest Cr(VI) removal is observed for the highest studied bio-char dosage of 5 g. The removal efficiency was 100 % and no Cr(VI) was detected for bio-char dosage of 3, 4, and 5 g after 5 hours of sorption reaction. After 3 hours of sorption reaction, the removal efficiencies were the same almost as 100 % for bio-char dosages of 4 and 5 g. Because of this similar removal, 4 g was considered as the optimum bio-char dosage. Other reports have also recommended similar optimum value for bio-char sorption [23, 24, 25, 31].

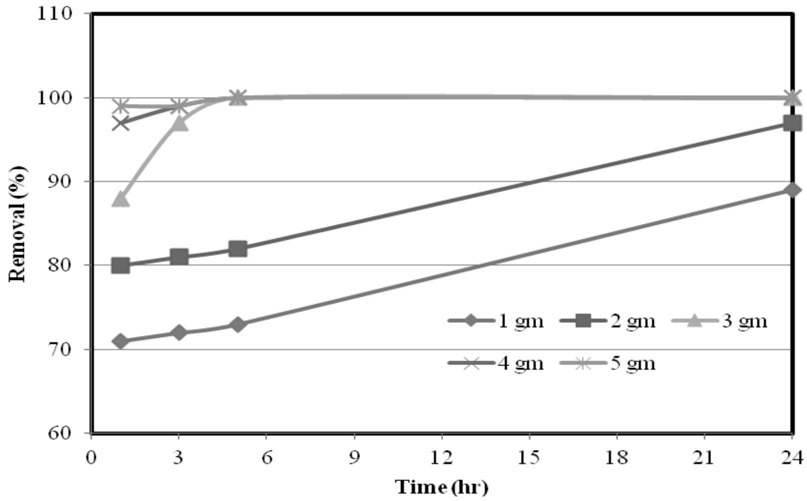
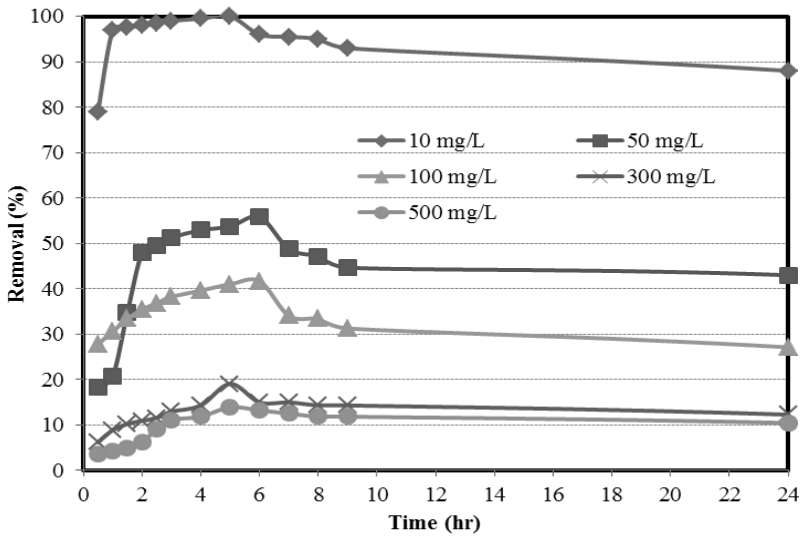


Figure 2: Removal efficiencies of Cr(VI) at various times with different dosages of bio-char as sorbent. pH: 2; solution volume: 200 mL; initial Cr(VI) concentration: 10 mg/L.

3.1.3 Effects of initial Cr(VI) concentrations

For all initial concentrations studied, the removal efficiencies increased until it reached a maximum and then gradually decrease with time until an equilibrium value was achieved after 2 to 6 days (Fig. 3). For an initial Cr(VI) concentration of 10 mg/L, the Cr(VI) removal efficiency was around 80 % after 30 min, and increased to 100 % at 5 hours, and then started to gradually decrease with time as discussed above. The removal efficiencies reached the peak at around 56 %, 41 %, 20 %, and 14 % for initial Cr(VI) concentrations of 50, 100, 300, and 500 mg/L, respectively. Therefore, 10 mg/L was chosen as the optimum initial Cr(VI) concentration. Other studies have also recommended similar optimum values [24, 25, 31].



Figire 3: Removal efficiencies of Cr(VI) at various times with different initial Cr(VI) concentration using bio-char as sorbent. pH: 2; solution volume: 200 mL; amount of bio-char: 4 g.

3.2 Evaluations of the sorption capacities

The sorption capacities (q_t) improved with an increase in the initial Cr(VI) concentrations until they reached sorption peaks and then started to decrease gradually with time until an equilibrium values (q_e) were achieved between 9 hours and 6 days (Fig. 4). The lowest sorption capacity of 0.248 mg/g was found after 5 hours for the lowest studied initial Cr(VI) concentration of 10 mg/L. However, the highest sorption capacity of 1.717 mg/g was obtained after 5 hours for the highest studied initial Cr(VI) concentration of 500 mg/L. Sorption capacities at a peak of 0.738, 1.150, and 1.470 mg/g were reached for initial Cr(VI) concentration of 50, 100, and 300 mg/L, respectively.

4. CONCLUSIONS

In this study bio-char was used to uptake Cr(VI) from synthetic wastewater. The optimum value of pH, bio-char dosage, and initial Cr(VI) concentration were found 2, 20 g/l, and 10 mg/l, respectively for the highest (100 %) sorption of Cr(VI) onto the bio-char. The maximum adsorption capacity of 1.717 mg/g was found after 5 hours for initial Cr(VI) concentration of 500 mg/l.

Since untreated bio-char was used in this study, it could be treated in the acidic medium prior to use in the sorption process. Synthetic wastewater that was used in this study did not contain other substances such as other metal ions, microorganism, and humic acid. In the future, real wastewater sample collected from the industries could be the promising research option. Bio-char should be free from chromium contamination.

REFERENCES

[1] Kabata-Pendias, A. and Mukherjee, A. B. (2007). “Trace Elements from Soil to Human”
Springer-Verlag Berlin Heidelberg, New York. ISBN-10 3-540-32713-4 and ISBN-13 978-3-540-32713-4.
[2] Daniel, R., Anjaneyulu, Y., and Krupadam, R. J. (2009). “Cr (VI) removal from electroplating industrial effluents: A greener and cheaper method”. Zastita Materijala, 50:13-18.
[3] Pugh, C. T., Palmer, T. T., and Carr, P. A. (2008) “A study of significant variables and their effects on the analysis of hexavalent chromium”. J. Chem. Health Saf., 15 (3), 25.
[4] Barceloux, D. G. & Barceloux, D., (1999). Chromium. Clinical Toxicology, 37(2): 173–194.
[5] Al-Jabari, M., Abualfailat, M., Shaheen, S. (2012). ‘Treating leather tanning wastewater with stone cutting solid waste”. Clean-soil, air, water, 40 (2): 206-210.
[6] C Pellerin and S M Booker. (2000). “Reflections on hexavalent chromium: health hazards of an industrial heavyweight”. Environ Health Prospect. 2000 September; 108(9): A402–A407.
[7] Kurniawan, T. A., Chan, G. Y. S., Lo, W. H., and Ba-bel, S. (2006). “Physico-chemical treatment techniques for wastewater laden with heavy metals, Chemical Engineering Journal, 118, 83–98.
[8] Roundhill, D. M., Koch, H. F. (2002). “Methods and techniques for the selective extraction and recovery of oxianions”. Chem. Soc. Rev. 31:60–67
[9] Mohan, D. & Pittman, C. U. (2006). “Activated carbons and low cost adsorbents for remediation of tri- and hexavalent chromium from water”. Journal of Hazardous Materials B137: 762–811.
[10] Mohan, D., Rajput, S., Singh, V. K., Steele, P. H., Pittman, C. U. (2011). “Modeling and evaluation of chromium remediation from water using low cost bio-char, a green adsorbent”. Journal of Hazardous Materials,188: 319–333.
[11] Tiravanti, G., Petruzzelli, D., Passino, R. (1997). “Pretreatment of tannery wastewaters by an ion exchange process for Cr(III) removal and recovery”. Water Sci. Technol. 36: 197–207.
[12] Rengaraj, S., Yeon, K. H., Moon, S. H. (2001). “Removal of chromium from water and wastewater by ion exchange resins”. J. Hazard. Mater. 87 (1–3): 273–287.
[13] Rengaraj, S., Joo, C. K., Kim, Y., Yi, J. (2003). “Kinetics of removal of chromium from water and electronic process wastewater by ion exchange resins: 1200H, 1500H and IRN97H”. J. Hazard. Mater. 102 (2/3): 257–275.
[14] Petruzzelli, D., Passino, R., Tiravanti, G. (1995). “Ion exchange process for chromium removal and recovery from tannery wastes”. Ind. Eng. Chem. Res. 34: 2612–2617.
[15] Rivero, M.J., Primo, O., Ortiz, M.I. (2004). Modelling of Cr (VI) removal from polluted groundwaters by ion exchange”. J. Chem. Technol. Biotechnol, 79:822-829.
[16] Kozlowski, C. A., Walkowiak, W. (2002). “Removal of chromium(VI) from aqueous solutions by polymer inclusion membranes”. Water Res. 36: 4870–4876.
[17] Shaalan, H., Sorour, M., Tewfik, S. (2001). “Simulation and optimization of a membrane system for chromium recovery from tanning wastes”. Desalination 14: 315–324.
[18] Barakata, M. A. and Schmidt, E. (2010). “Polymer-enhanced ultrafiltration process for heavy metals removal from industrial wastewater”. Desalination, 256: 90-93.
[19] Ozaki, H., Sharmab, K. and Saktaywirf, W. (2002). “Performance of an ultra-low-pressure reverse osmosis membrane (ULPROM) for separating heavy metal: effects of interference parameters”. Desalination, 144: 287-294.
[20] Akbal, F. and Camcl, S. (2011). “Treatment of metal plating wastewater by electrocoagulation”. Environmental progress & sustainable energy, 31 (3) 340-350
[21] Li, Y., Zeng, X., Liu, Y., Yan, S., Hu, Z., & Ni, Y. (2003). “Study on the treatment of copper-electroplating wastewater by chemical trapping and flocculation”. Separation and Purification Technology, 31: 91–95.
[22] Martinez, S.A. & Rodriguez, M.G. (2007) “Dynamical modeling of the electrochemical process to remove Cr (VI) from wastewaters in a tubular reactor”. J. Chem. Technol. Biotechnol, 82: 582-587.

[23] Zhonghua, H., Lin, L., Yijiu, L., Yaming, N. (2003). "Chromium Adsorption on High-Performance Activated Carbons from Aqueous Solution". Sep. Purif. Technol. 31(1),13.
[24] Mohan, D., Singh, K. P., Singh, V. K. (2005). "Removal of hexavalent chromium from aqueous solution using low-cost activated carbons derived from agricultural waste materials and activated carbon fabric cloth". Ind. Eng. Chem. Res. 44: 1027–1042.
[25] Mohan, D., Singh, K. P., Singh, V. K. (2006), "Trivalent chromium removal from wastewater using low cost activated carbon derived from agricultural waste material and activated carbon fabric cloth" J. Hazard. Mater. 135: 280–295.
[26] Yu, L. J., Shukla, S. S., Dorris, K. L., Shukla, A., Margrave, J. L. (2003) "Adsorption of Chromium from Aqueous Solution by Maple Saw Dust". J. Hazard. Mater. 100 (1–3): 53.
[27] Chakir, A., Bessiere, J., Kacemi, K. E., Marouf, B. (2003). "A Comparative Study of the Removal of Trivalent Chromium from Aqueous Solutions by Bentonite and Expanded Perlite". J. Hazard. Mater. 95 (1–2): 29.
[28] Mcafee, B. J., Gould, W. D., Nadeau, J. C., Costa, A. C. A., (2001). Bio sorption of metal ions using chitosan, chitin, and biomass of rhizopus oryzae. Separation Science & Technology, 36(14): 1520.
[29] Lakatos, J., Brown, S. D., Snape, C. E. (2002). "Coals as sorbents for the removal and reduction of hexavalent chromium from aqueous waste streams". Fuel 81: 691–698.
[30] Yuan, P., Liu, D., Fan, M., Yang, D., Zhu, R., Ge, F., Zhu, J., He, H. (2010). "Removal of hexavalent chromium [Cr(VI)] from aqueous solutions by the diatomite-supported/unsupported magnetite nanoparticles". Journal of Hazardous Materials, 173: 614–621.
[31] Dong, X., Ma, L. Q., Li, Y., (2011). Characteristics and mechanisms of hexavalent chromium removal by bio-char from sugar beet tailing. Journal of Hazardous Materials, 190: 909-915
[32] Langmuir, I. (1916). "The constitution and fundamental properties of solids and liquids". Part I. Solids. J. Am. Chem. Soc. 38, 2221.
[33] Freundlich, H.M.F. (1906). "Uber die Adsorption in Losungen". Z. Phys. Chem. 57(A), 385.
[34] Lagergren, S. (1898). About the theory of so-called adsorption of soluble substances. Kungliga Svenska Vetenskapsa-kademiens. Handlingar, 24(4):1-39
[35] Ho, Y. S., McKay, G. (1998). "Pseudo-second order model for sorption processes". Process Biochemistry 34 (1999) 451.

ASSESSMENT OF GREY WATER USE POTENTIAL: A CASE STUDY

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ABSTRACT

Bangladesh is one of the most densely populated countries in the world. Due to rapid population growth and unplanned urbanization, the scarcity of fresh water is one of the major concerns of basic utilities. Considering this aspect there is always a need to critically look at possible approaches to ensure water availability for its intended uses while avoiding overexploitation of today's freshwater sources. The approaches are already identified those include rainwater harvesting, wastewater reuse and desalination. In this study the potential of grey-water use, one of the many ways under wastewater recycle & reuse was studied in a small scale for non-potable purposes, where quality may be compromised based on desired uses. This study is also carried out to investigate the potential of grey water for various non potable uses that can substitute and supplementing fresh water supply. For this study 'Sufia Kamal Hall' at CUET in Chittagong was selected as a primary study site. Grey water from different sources, such as kitchen water, bathroom & wash basin, mixed water apart from water from toile flushing. To make water fit for different non potable uses, collected water was pass through a single pass sand filter that was designed and prepared by locally available materials. Grey- water from the three sources was characterized first and then was treated through this filter. Comparison was made to evaluate its increased use potential for non-potable purposes. By observing and comparing the results of treated and untreated grey water, it can be inferred that this water can be made reusable with minimum treatment obtained from the designed single pass sand filter, which are not only cheapest option but also cost-effective, as the materials used are all locally available. Considering the quality of effluent, it is found that this wastewater treated can be used for non potable uses such as irrigation, recreational purpose, fish culture, construction purpose and industrial purposes due to its low contamination. The reuse of grey water is therefore seen as an attractive solution of minimizing the deficit between demand & supply of fresh water, where quality can be compromised. It is hoped that this study will open the window of grey water potential thus in turn reduce pressure on fresh water uses.

Keywords: Grey-water, Non-potable uses, Sand filter, Water quality, CUET.

1. INTRODUCTION

Bangladesh is world's eighth most populous country with a population of more than 160 million people in a territory of 56,977 sq mi (BBS 2010). The WHO estimates that 97% of the people of Bangladesh have access to water and only 40% percent have proper sanitation (Pal et al., 2010). With a staggering, approximately 60% of the population has to endure unsafe drinking water, the nation is in danger. The statistics even did not reflect the field scenario (Pal et al., 2010). People of many parts of Bangladesh suffer from water crisis all the year around. With the bulging of the population, demand for water is growing fast but the supply of water does not increase accordingly. The sources of freshwater used today are in threats due to over exploitation and pollutants load from untreated discharge of wastes from different sources. To reduce the pressure on freshwater sources and stop using freshwater where it may not mandatory, such as for irrigation, fish culture, gardening, recreation, groundwater recharge etc., waste water use or reuse can be an attractive solution (Al-Jayyousie, 2003; Redwood, 2007). Within wastewater, grey water is the component of domestic wastewater, which has not originated from the toilet or urinal rather generated from water used in hand washing, bathing and are around 50-60% of total grey water which include kitchen water as well. The wastewater collected from wash basin and bathing is considered to be the least contaminated type of grey water. Common chemical contaminants include soap, shampoo, hair dye, toothpaste and cleaning products (Redwood, 2007). It is unlikely but may not be unexpected if it contains faecal coliform (and the associated bacteria and viruses). Water used in cloth washing generates around 25-35% of total grey water. Grey water generated due to cloth washing can have fecal contamination with the associated pathogens and parasites such as bacteria. Kitchen grey water contributes about 10% of the total grey water volume. It is contaminated with food particles, oils, fats and other wastes. It readily promotes and supports the growth of micro-organisms. Therefore kitchen wastewater may not be well suited for use without a minimum level of treatment compared to other sources in all types of grey water systems (Almedia et al., 1999; Butler et al., 1995; Birks and Hills, 2007).

In terms of basic water quality parameters for non potable uses (TSS, TDS, BOD, Turbidity, FC), grey water is considered to be comparable to a low-or medium grade wastewater. Jefferson et al. (2004) found that, though similar in organics content to full domestic wastewater, grey water tends to contain fewer solids and is less turbid than full domestic wastewater, suggesting that more of its contaminants are dissolved. Subsequently grey water reuse in many parts of the world, including both developed and developing countries, has gained significance for last few decades. Many investigations have been conducted on domestic grey water quality analysis, treatment and reuse in the EU, USA, Middle-east countries, Japan and Australia (Al-jayyousie, 2003; Eriksson et al., 2002; Friedler and Hadari, 2006; Butler et al., 1995; Birsk and Hills, 2007). Grey water treatment systems have been successfully implemented in the US, Japan and Australia to reclaim grey water for non-potable uses, as found from the above studies.

Though grey water composition varies widely from household to household, depending on the personal habits of residents and the products used in the home, and also vary according to source: each fixture contributing to the grey water collection system will carry its own particular contaminant load, this part of wastewater could provide the most effective use potential with or without treatment depending on its intended purposes. Within the context of this background, it can be said that grey water seems to be a potential alternative of fresh water saving. The ladies hall, which is named as Sufia Kamal Hall, Chittagong University of Engineering & Technology (CUET), Chittagong is selected as study area. The main objective of this project is to evaluate grey water potentiality for non potable uses. Under this, the sub objectives which are identified are to characterize grey water & to see its potentiality for non potable use such as irrigation, concrete mixing, recreational purpose and fish culture, to design a cost effective and efficient grey water treatment system & to improve raw water quality so as to make it fit for non potable uses.

2. MATERIALS & METHODS

2.1 Sample Collection

9 samples were collected from Sufia Kamal Hall. Three different sources were selected. One was kitchen water sample, second was washing application which include water from shower, wash basin, cloth washing. The third sample was the mixed sample of kitchen water and water from wash application. Three samples from each source were collected during sample collection. The samples were collected weekly basis for a month period. The kitchen water samples were collected from 11.30 am to 1 pm while other samples were collected from 8am to 1pm. The samples were collected in plastic bottles. The bottles were cleaned with hot water and distilled water before collecting samples to avoid contamination. The samples were collected from the outlet of kitchen while for shower and wash basin after use and before entering into the outlet. The mixed wastewater was collected from the outlet drain.

2.2 Sample Preservation

Storage of grey water prior to reuse is discouraged because it can affect the pathogen load of both raw and treated grey water. Dixon et al. (2000) tested a model for predicting quality changes in stored grey water, based on observed processes of settlement of suspended solids, aerobic microbial growth, anaerobic release of soluble COD from settled organic matter, and atmospheric re-aeration. The study suggests that storage of grey water for 24h could potentially improve water quality, but storage for more than 48h could seriously deplete dissolved oxygen (DO) levels and lead to what they call “aesthetic problems”, including anaerobic processes and associated smells. Hence, for this study, the samples were preserved not more than 6 hours after collection. The samples were preserved on fridge if not analyses immediately after collection.

2.3 Sample Analysis

The untreated and treated grey water were characterized primarily for pH, Color, Turbidity, Salinity, TS, TDS, TSS, COD, BOD_5, Nitrate, Phosphate, Hardness, Alkalinity, Total Coliform and Fecal Coliform. The treated and untreated grey water were analyzed by using standard methods prescribed by American Public Health Association and Water Pollution Control Federation, Standard Methods for the Examination of water and wastewater (APHA, 2003; Metcalf and Eddy, 2007). The laboratory test was carried out in the 'Environmental Engineering Laboratory' at CUET. In all the measurement quality control and quality assurance were maintained as per laboratory manual.

2.4 Design Procedure of Single Pass Sand Filter

A water filter removes impurities from water by means of a fine physical barrier, a chemical process or a biological process. This study adopted single pass sand filter using locally available materials. In a single pass sand filter wastewater passes vertically downwards through a bed of sand and gravel in a largely aerobic environment which provides excellent conditions for BOD_5 removal, and nitrification (Birks and Hills, 2007). In this study a single pass sand filter was designed for treating grey water with the concept developed by Headley (2007) (National Institute of Water & Atmospheric Research). The detail of this filter along with specification and schematic (see Fig. 1) is discussed below:

- Materials: Local sand, gravel and pea gravel. The filter was prepared in a clean plastic drum of height 1m. 10 concrete cubes and square steel net were used over which the layer of sand and gravel were provided. The schematic diagram can be seen in Fig1.
- The filtration rate of this filter was measured as 0.97 to 1.07 liter/min initially.
- Material Specification: The sand used should be durable, have a coarse (large) and fairly uniform grain size (judged by the d10 and uniformity coefficient) and be relatively clean (as judged by the % fines), as suggested by Headley (2007). The technical specifications as follows:
  Effective size (d10) =0.25 – 0.75 mm
  Uniformity Coefficient (d10 / d60) < 4
  Percent fine material < 5%, F.M = 2.5-3.50
  Gravel of size 10mm, Pea Gravel of size 5mm
- Laboratory test were carried out for selecting materials of specified sizes.

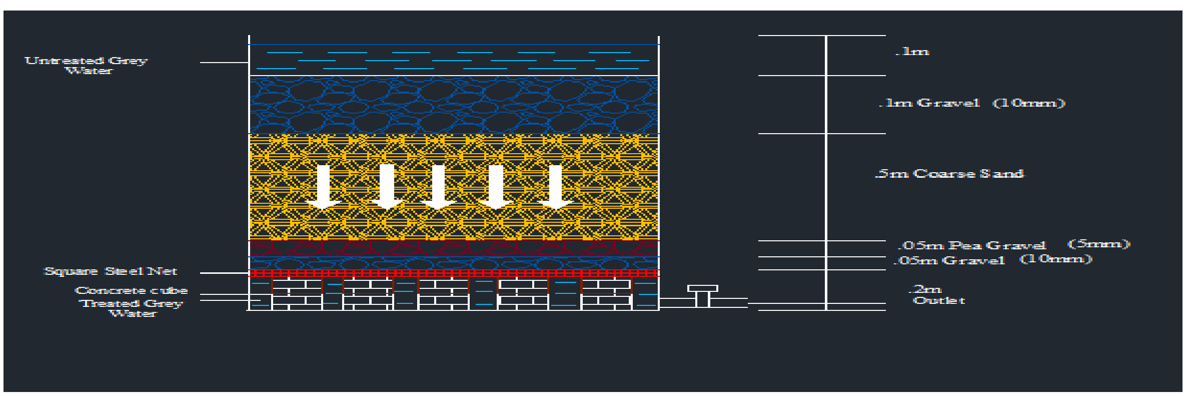

Fig 1: Cross-Sectional Profile of Single Pass Sand Filter (after Headley, 2007)

The drum was washed first with fresh water. Two layers of concrete cube were provided, 5 in each layer. Over this layer square steel net was given. These cubes helped the steel net to support the various media layer. A water tap was placed in the drum which was used to collect filtered water. Over the net, at first 0.05 m of gravel layer was given. A layer of 0.05 m of pea gravel was provided over it. Then a thick layer of coarse sand was provided. The thickness of this layer was 0.50 m. At the last stage, a gravel layer of 0.10 m was given. Before using the materials for preparing filter, these were washed by fresh water repeatedly and then dried in air. The treated water was collected from the tap located at the bottom of the designed filtration tank as shown in Fig1.

3. RESULTS AND DISCUSSION

3.1 Characterization of Grey Water

Water quality parameters of untreated grey water samples were tested by the standard method (APHA 2003). The results are presented in Table 1. Kitchen water samples which were tested showed pH less than 7 were said to be acidic. But the tested samples of washing application were alkaline due to the presence of soaps and detergents. The mixed samples showed the cases which were similar to kitchen water sample. The cloudiness or the haziness of the kitchen water samples were more than the samples of washing application due to the presence of food components.

Table 1: Characteristics of grey water originated from different sources

Water quality parameters with units	Kitchen			Bathroom+ washbasin			Mixed water		
	Sample 1	Sample 2	Sample 3	Sample 1	Sample 2	Sample 3	Sample 1	Sample 2	Sample 3
p^H	6.10	***3.60***	4.80	**8.85**	7.08	7.8	5.65	6.3	5.80
Turbidity(NTU)	869	957	**910**	***157***	184	175	593	620	600
Color(PCU)	ND	ND	ND	***35***	**45**	40	ND	ND	ND
Total Solid(mg/l)	6600	**8100**	6800	***650***	1930	1290	2190	3850	3020
Total dissolved solid(mg/l)	3620	**4410**	4000	***540***	1750	1140	1870	2740	2300
Total suspended solid(mg/l)	2980	**3690**	3335	***110***	180	140	320	1110	720
Salinity(mg/l)	80	70	75	***25***	100	64	66	**200**	135
COD(mg/l)	**800**	600	700	***24***	60	42	420	600	510
BOD_5 (mg/l)	54	72	63	***20***	48	35	**108**	90	100
Alkalinity(mg/l)	264	280	272	340	***240***	300	300	**400**	360
Hardness(mg/l) as $CaCO_3$	260	280	260	***250***	***250***	***250***	320	300	**350**
Nitrate(mg/l)	ND	ND	ND	***20***	***20***	***20***	**30**	ND	ND
Phosphate(mg/l)	ND	ND	ND	**2**	*1*	*1*	ND	ND	ND
Total coli forms (count/100ml)	900	600	800	TNTC	TNTC	TNTC	TNTC	TNTC	TNTC
Fecal coli forms (count/100ml)	400	500	400	TNTC	TNTC	TNTC	TNTC	TNTC	TNTC

Note: ND: Not Detectable; TNTC: Too Numerous To Count;
Values shown in 'bold' and 'bold & Italic' indicate highest and lowest concentration respectively

The mixed samples showed the results within the samples of kitchen water and washing application. The visual perceptual property of the samples which was defined by color was not detectable for kitchen water and mixed samples due to the higher concentration of suspended food particles. But the color of the samples from washing application was much lighter than the others as it did not contain any food particle. The amount of total solid was much higher in kitchen water. By observing the concentration of total dissolved solid of the samples, it can be said that the kitchen water sample contained more impurities or more inorganic salts than the washing sample and the mixed wastewater. The total suspended solids, including the volatile fraction (VSS) were commonly monitored to evaluate the degrees of pollution in water (Almedia et al., 1999). Comparing all 9 samples from different sources and combined sources, as seen in Table 1, it can be said that the kitchen water were more polluted due to the presence of food waste. The salinity or the dissolved salt content of the mixed sample was more than the other samples as it contained food particles, soaps and detergents. The amount of COD was more in kitchen water sample and in mixed sample due to containing more organic matters than the water from washing application. The value of BOD5 was higher for kitchen water and mixed water samples because the aerobic biological organisms were more likely present in these wastewater, which increased the biodegradable activity. From the result of alkalinity, it can be said that the quantitative capacity of mixed water sample to neutralize acid was higher than other samples.

Table 2: Characteristics of treated grey water

Water quality parameters with units	Kitchen		Bathroom+ washbasin		Mixed	
	Sample 1	Sample 2	Sample 1	Sample 2	Sample 1	Sample 2
p^H	7.58	6.60	6.20	6.51	7.40	7.56
Turbidity(NTU)	45	50	40	45	46.9	52
Color(PCU)	30	35	20	25	35	35
TS(mg/l)	1040	1100	790	798	1310	1400
TDS(mg/l)	1000	1060	700	710	1270	1300
TSS(mg/l)	40	40	90	88	40	100
Salinity(mg/l)	40	45	16	20	25	30
COD(mg/l)	120	160	20	25	110	130
BOD_5 (mg/l)	40	44.5	6	8	36	42
Alkalinity(mg/l)	140	165	100	120	260	300
Hardness(mg/l as $CaCO_3$	252	255	120	120	320	327
Nitrate(mg/l)	10	10	10	10	10	Nil
Phosphate(mg/l)	1	1	Nil	Nil	2	2

Hard water requires more soap and synthetic detergents for home laundry and washing. Hard water is that which contains high mineral content. By observing data of hardness (see Table 1), it is clear that the samples of water from various sources were very hard due to the presence of iron. The mixed water sample shows slight higher value than others as it contained more soaps and detergents on top of iron. Nitrate concentration of the tested samples of washing application indicates the presence of small absolute quantities of fecal matter. Due to high concentration of suspended solids it was not possible to detect nitrate concentration in kitchen water and mixed water samples. Phosphate concentration of the tested samples of the washing application indicates the non hazardous effect of water due to the use of phosphate free soaps. In kitchen water and mixed water samples, this concentration was not detected. By considering total coliform count per 100ml, it can be said that the samples had a high probability of contamination by protozoa, viruses and bacteria. The results obtained from the different sources were found to be consistent with the data presented elsewhere (Butler et al., 1995; Almedia et al., 1999; Angelakis and Bontoux, 2001).

Table 3: Comparison between Treated and Untreated Grey Water

Water quality parameters with units	Kitchen		Bathroom+ washbasin		Mixed		Change +/_
	Untreated	Treated	Untreated	Treated	Untreated	Treated	
p^H	4.8	**7.09**	7.91	**6.36**	5.92	**7.48**	+
Turbidity(NTU)	912	**47.5**	172	**42.5**	604.33	**49.45**	_
Color(PCU)	ND	**32.5**	40	**22.5**	ND	**35**	_
Total Solid(mg/l)	7167	**1070**	1290	**794**	3020	**1355**	_
Total dissolved solid(mg/l)	3335	**1030**	1143.33	**705**	2303	**1285**	_
Total suspended solid(mg/l)	4010	**40**	143	**89**	716.67	**70**	_
Salinity(mg/l)	75	42.5	63	18	133.67	27.5	_
COD(mg/l)	700	**140**	42	**22.5**	510	**120**	_
BOD_5 (mg/l)	63	**42.25**	34	**7**	99.33	**39**	_
Alkalinity(mg/l)	272	**152.5**	293.33	**110**	353.33	**280**	_
Hardness(mg/l as $CaCO_3$	266.67	253.5	250	120	323.33	323.5	_
Nitrate(mg/l)	ND	**10**	20	10	ND	**10**	_
Phosphate(mg/l)	ND	**1**	1	Nil	ND	**2**	_

Note: (+) sign indicates increase and (-) sign indicates decrease.

3.2 Characterization of Filtered Grey Water

The raw wastewater collected from the different sources was passed through a single pass sand filter designed for this study to characterize the filtered water. The result obtained is presented in Table 2. Moreover, comparison was made to evaluate the degree of change this filter offered as presented in Table 3. In Table 3 the mean values were presented. It has been found that except pH, all other parameters were found to decrease moderate to a significant degree. For example, due to filtration the particulate matter was removed from water and hence turbidity and concentration of TSS were drastically reduced. Similarly, the organic matters were removed by the layers of sand and gravel, which in turn lowered the concentration of COD and BOD_5 in filtered water. The layer of sand present is believed to be reduced the concentration of soaps and detergents (Headley, 2007). Due to low contamination of grey water, the water can be treated up to a certain range through a single pass sand filter with ease. It has been observed that filtration rate as seen initially was not changed much after passing the all the samples collected during the study signifying that filter maintenance may not need to done very frequently.

3.3 Suitability of Untreated Grey Water for Non Potable Uses

Due to low contamination of filtered grey water, it is anticipated that this water can be used in various non potable uses, such as irrigation, fish culture, recreation, gardening, concrete mixing etc. where quality may be compromised.

Table 04- Suitability of Untreated Grey Water for Non Potable Uses

SL NO.	Parameter with unit	Standard	Standard	Standard	Standard	Untreated	Untreated	Untreated	Remark
		Irrigation purpose (water quality guideline)	Recreational purpose (ECR 97)	Concrete Mixing (EN 1008)	Fish culture (ECR 97)	Kitchen water (mean)±Standard Deviation	Bathroom & wash basin (mean) ± Standard Deviation	Mixed Sample (mean)±Standard Deviation	
1	pH	6.5-8.4	6.5–8.5	6.0-8.0	6.5 – 8.5	4.8±1.25	7.91±0.89	7.48±0.11	water from bathroom and wash basin can be used only
2	Turbidity (NTU)	NA	50	NA	NA	912±44.03	172±13.75	604.333±3.61	not ok
3	Color(PCU)	NA	NA	NA	NA	ND	40±3.54	ND	not ok
4	Chloride (mg/l)	100	NA	500	NA	75±5	63±37.51	133.667±67.01	ok
5	Coliform(Total) count / 100 ml	1	200 or less	NA	5000 or less	770±157.75	TNTC	TNTC	only kitchen water can be used for fish culture & recreational purpose
6	Coliform(Faecal) count / 100 ml	1	200 or less	NA	5000 or less	450±57.75	TNTC	TNTC	only kitchen water can be used for fish culture & recreational purpose
7	Nitrate (mg/l)	NA	NA	NA	NA	ND	20±0	ND	not ok
8	Phosphate (mg/l)	NA	NA	NA	NA	ND	1±0.58	ND	not ok
9	Hardness (mg/l)	NA	NA	NA	NA	266.67±11.55	250±0	323.33±25.17	not ok
10	TSS(mg/l)	50	NA	2000	NA	3335±355	143±54.70	716.667±395.01	washing application and mixed water samples can be used only for concrete mixing
11	TDS(mg/l)	40	NA	2000	NA	4010±395.09	1143.33±618.51	2303±435.01	only samples of washing appicatin can be used for concrete mixing
12	COD(mg/l)	NA	NA	NA	NA	700±100	42±18	510±90	not ok
13	BOD_5 20°C(mg/l)	NA	3 or less	NA	6 of less	63±9	34±14.01	99.333±9.09	not ok

The mean concentration present for each parameter tested for raw water and filtered water was then presented along with standard values available for non potable purposes in Table 4 and Table 5. Considering Table 4, it is seen that most of the parameters exceed one or more standard values required to qualify water to be used for its intended uses. For example, kitchen water can be used directly for fish culture if sensitive fish species are not cultured, while basin water and shower water can be used to prepare concrete mix only. Apparently presence of high concentration of suspended solid and coliform in raw water, wastewater without treatment cannot be fit for several non potable uses.

3.4 Suitability of Treated Grey Water for Non Potable Uses

Table 05- Suitability of Treated Grey Water for Non Potable Uses

SL NO.	Parameter with unit	Standard Irrigation purpose (water quality guideline)	Standard Recreational purpose (ECR 97)	Standard Concrete Mixing (EN 1008)	Standard Fish culture (ECR 97)	Treated Kitchen water (mean)±Standard Deviation	Treated Bathroom & wash basin (mean)±Standard Deviation	Treated Mixed Sample (mean)±Standard Deviation	Remark
1	pH	6.5-8.4	6.5–8.5	6.0-8.0	6.5 – 8.5	7.09±0.69	6.36±0.22	7.48±0.11	kitchen water and mixed water samples can be reusable
2	Turbidity (NTU)	NA	50	NA	NA	47.5±2.5	42.5±3.54	49.45±3.66	samples can be reused for recreational purpose only
3	Color(PCU)	NA	NA	NA	NA	32.5±3.54	22.5±3.54	35±0	not ok
4	Chloride (mg/l)	100	NA	500	NA	42.5±2.5	18±2.83	27.5±3.54	ok
7	Nitrate (mg/l)	NA	NA	NA	NA	10±0	10±0	10±0	not ok
8	Phosphate (mg/l)	NA	NA	NA	NA	1±0	Nil	2±0	not ok
9	Hardness (mg/l)	NA	NA	NA	NA	253.5±2.12	120±0	323.5±4.96	not ok
10	TSS(mg/l)	50	NA	2000	NA	40±0	89±1.41	70±42.43	washing application and mixed water samples can be reusable for concrete mixing but failed to maintain irrigation standard
11	TDS(mg/l)	40	NA	2000	NA	1030±30	705±7.07	1285±21.21	samples can be reused for concrete mixing but failed to maintain irrigation standard
12	COD(mg/l)	NA	NA	NA	NA	140±28.28	22.5±3.54	120±14.14	not ok
13	BOD_5 20°C(mg/l)	NA	3 or less	NA	6 of less	42.25±3.18	7±1.41	39±4.24	bathroom & wash basin is marginally above the standard for fish culture while others do not qualify

In contrast to suitability of raw water (Table 4), filtered water through single pass sand filter, it possessed more pertinence for reuse application. Due to the reduction in the concentration of all the parameters effluent obtained are compared with the standard requirements and it has been seen that water is now appears to be suitable for all purposes except high BOD_5. Even though higher BOD loading, still water is not as bad as it was initially. More trails are going on to lower the BOD load by applying chlorine.

4. CONCLUSION:

By observing this study, the following points are marked:

- ❖ Based on raw water analysis it has been found that the concentration of TSS (3335 mg/l), TDS (4010 mg/l), Hardness (323.33 mg/l), Color (ND), Turbidity (604.333 NTU), TC (TNTC count per 100 ml), FC (TNTC count per 100 ml), BOD of 5 days (99.333 mg/l), COD (700 mg/l) exceeded values provided for non potable uses.
- ❖ In filtered grey water the concentration of priority pollutants are TSS (40 mg/l), TDS (1030 mg/l), Color (32.5 PCU), Turbidity (47.5 NTU), BOD of 5 days (42.5 mg/l), COD (140 mg/l). Due to filtration the concentration of the pollutants have reduced which satisfies the standards provided for non potable uses such as irrigation, recreational purpose, concrete mixing and fish culture.
- ❖ The results show higher levels of pathogenic organisms which can posses a real health risk o human thus not suitable for any use where human contact may comes in.

So that it can be concluded that grey water can be made reusable for non potable uses with this level of treatment. Further studies are needed to minimize pathogenic organism.

REFERENCES:

Al-Jayyousie O.R., (2003) Grey water reuse: towards sustainable water management. Desalination 156(1), 181-192.

Almeida, M. C., Butler, D., & Friedler, E., 1999. At-source domestic wastewater quality. Urban Water, 1(1): 49–55.

Angelakis A.N. and Bontoux L., (2001) Wastewater reuse and reclamation in European Countries. Water Policy 3, 47-59.

A. Khatun & M.R. Amin, Bangladesh University of Engineering and Technology, (2011) Dhaka, Bangladesh. Grey water reuse: a sustainable solution for water crisis in Dhaka, Bangladesh.

American Public Health Association and Water Pollution Control Federation, Standard Methods for the Examination of water and wastewater 2003.

Butler, D., Friedler, E.,and Gatt, K.,1995. Characterizing the quantity and quality of domestic wastewater inflows. Water Sci. Tech.31(7): 13-24.

Birks, R., & Hills, S. (2007). Characterisation of Indicator Organisms and Pathogensin Domestic Greywater for Recycling. Environmental Monitoring and Assessment, 129(1-3), 61-69.

Eriksson, E., Auffarth, K., Henze, M. and Ledin, A., 2002. Characteristics of grey wastewater. Urban Water. 4: 85–104.

EN 1008 Mixing water for concrete – specification for sampling, testing and assessing the suitability of water, including water recovered from processes in the concrete industry, as mixing water for concrete.

Jefferson, B., Palmer, A., Jeffrey, P., Stuetz, R., Judd, S. (2004). Grey water characterization and its impact on the selection and operation of technologies for urban reuse. Water Science and Technology, 50(2), 157-164.

Redwood, M., 2007. Greywater Use in the Middle East and North Africa Region. Background Paper for IDRC-CSBE Experts Meeting, Jan, 2007.

Metcalf and Eddy, Inc. , 2007. Water Reuse: Issues, Technologies, and Applications. New York, The McGraw Hill Company.

Tom Headley (National Institute of Water & Atmospheric Research) On-site Sewage Treatment Using Single-Pass Sand Filter: -Case Study Design for 5 Ora Street, Gisborne

A PRELIMINARY SURVEY OF RESIDENTS FOR THE PROJECT TO PREVAIL TSS IN PARBAYARJHAPA, BANGLADESH

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ABSTRACT

Water quality is not good in Parbayarjhapa of Bangladesh. Therefore the survey of the life style, knowledge about health, health status, attitude to water was carried out with questionnaire in July of 2012. The number of subjects was 10. The questionnaire was made by 2 Japanese and 1 Bangladeshi. In summary, life style, knowledge about health, health status, attitude to water in Parbayarjhapa were not desirable from the viewpoint of human health. For example, Their frequency was considerably high (Once or more per month: 9 residents, Once or more per year: 1 resident). Their knowledge inclined to arsenic. The chloride ion concentration of pond water was 6750 mg/L and the concentration of ground water was 440 mg/L in Parbayarjhapa. Hearing survey from residents about life style and health was conducted in July of 2013. Some stated that those engaged in prawn culture, especially female workers, had inflammation of reproductive organ. Any solution suitable in Parbayarjhapa to water problem should be introduced. Because knowledge of water of residents may incline to arsenic, education concerning other than arsenic, such as salt and bacteria, is necessary for better health.

Keywords: Water, Salt, Tubular Solar Still, Diarrhea, Infectious Disease, Parbayarjhapa

1. INTRODUCTION

It is globally known that Bangladesh has water problem about arsenic contamination. However little is known about another water problem such as bacteria, salinity and so on. According to World Health Organization (WHO), under-five mortality rate (probability of dying by age 5 per 1000 live births) in Bangladesh was decreased from 84 in 2000 to 46 in 2011, while it was still higher than the average in upper-middle income countries (the probability in 2011: 20) and high income countries (the probability in 2011: 6). Diarrhoeal disease is one of major causes of child death in Bangladesh. Although the number of children who died of diarrhoea was also decreased, diarrhoea accounted for no less than 6% of death among children aged < 5 years in 2010. Meanwhile the proportion in upper-middle income and high income country were 4% and 1% respectively [WHO, 2013]. Because diarrhea is one of disease caused by bacteria [Stauber, 2012], unsanitary water containing bacteria may raise under-five mortality rate in Bangladesh.

It is well known that salt intake is related to various severe disease such as stomach cancer [Tsugane, 2004], heart disease [Tuomilehto, 2001], stroke (brain infarction) [Nagata, 2004], diabetes mellitus [Feldstein, 2002] and so on. Salinity in drinking water is also serious problem in coastal area of Bangladesh. As major resources of drinking water are ground water, pond water and river water, they are affected by saltwater intrusion from the Bay of Bengal. The intrusion is supposed to be caused not only by increase in natural disasters such as cyclone and massive floods resulting from climate change, but also by drawing saline water for prawn culture.

Actually many people have died of cardiovascular disease and diabetes; WHO reported that the age-standardized adult mortality rate in Bangladesh (ages 30-70, per 100 000 population) was 421 in 2008, the rate in upper-middle income and high income country were 295 and 104 respectively. The rate in South-East Asia Region, which was defined by WHO and included Bangladesh, was 322 [WHO, 2013]. Cardiovascular disease and diabetes in Bangladesh may be closely related to saltwater intrusion from the Bay of Bengal.

It would be impossible to stop climate change. As for drawing saline water, because prawn farming is one of major industries and indispensable for Bangladesh, it would be also very difficult to abandon the culture and saltwater intrusion will expand continuously from now on. That is why, it is not a problem that can be easily solved. Introduction of an apparatus which distillate saline water is realistic and necessary. Tubular Solar Still (TSS) has been developed as a distillation equipment and modified to fit in with coastal area of Bangladesh. Parbayarjhapa village, which is belonging to Khulna division and located at a southern part of Bangladesh, is affected by cyclone and massive floods. The major industry is prawn culture. Hence, Parbayarjhapa is nominated for TSS trial area. Prior to TSS introduction, the pilot survey was conducted in Parbayarjhapa. The residents' life style, knowledge about health, health status, attitude to water and so on were studied by hearing and questionnaire survey.

2. METHODOLOGY

The survey was conducted by means of fieldwork method such as inspection, interview and questionnaire on July 16 and 17, 2012. Subjects were 10 residents in Parbayarjhapa village; the number of male subjects was 5 and the age ranged from 14 to 55, the number of female subjects was 5 and the age ranged from 14 to 62. The questionnaire was made by 2 Japanese and 1 Bangladeshi. As the questionnaire was prepared in English, it was translated into Bengali at the actual survey in Parbayarjhapa village. The questionnaire was composed of items of subject characteristics, health status about diarrhoea, life style, water usage and attitude to water. Specific items of the questionnaire is mentioned in Table 1-4. The concentrations of chloride ion of the pond water and well water were measured at that time.

Hearing from residents about life style and health was conducted also in July of 2013.

The research protocol and questionnaire were approved by the ethics committee of School of Medicine, University of Fukui, Japan.

3. RESULTS

Table 1 shows water-use custom in Parbayarjhapa. Nobody had used pond water for drinking, while half of the subjects had used pond water for cooking. Pond water had been utilized for washing hands after lavatory and before eating meal. Some had used stored rainwater for drinking, cooking, washing hands before meal. The container for storage was coarse and insanitary (Figure 1). The size of the container observed was about 65 cm in diameter and about 30 cm in depth. Habit of washing hands after using lavatory and before eating was established, while the water for wash was pond and/or well water. Their way using mainly pond water might have not been sanitary.

The average time spent for carrying water per day was 32 min (5 min: 1 subject, 15 min: 5 subjects and 60 min: 4 subjects). All of subjects had diarrhoea and the frequency was considerably high (Once or more per month: 9 subjects, Once or more per year: 1 subject). There was no PSF in Parbayarjhapa.

Table 2 shows washing material used in Parbayarjhapa. The ratio of "Nothing" to "Soap" is 1 to 1 after lavatory and before eating meal, while most use soap for washing body.

Table 3 shows knowledge of harmful substances in water. All of subjects knew arsenic and salt. Among 7 subjects, who knew arsenic was harmful, 4 subjects knew skin problem as a specific effect of arsenic (Data not shown in Table 3). However among 8 subjects, who knew salt was harmful, nobody knew any specific harmful effect of salt (Data not shown in Table 3). Only half of subjects knew bacteria.

Table 4 shows subjects' attitude to water safety. They have generally been anxious about water and wanted safer water. However 3-4subjects had no anxiety about any water. They were less conscious to bacteria compared to arsenic and salt.

The concentrations of chloride ion of the water samples were rather high; the concentration of pond water was 6750 mg/L and the concentration of well water was 440 mg/L in Parbayarjhapa (cf. WHO reference value: 250 mg/L).

According to hearing in 2013, some complained of skin inflammation, others, especially women, complained of inflammation of sexual organ.

Table 1: Water-use (Number of subjects)

Questions	Pond water	PSF-filtered water	Tube-well water	Stored rainwater	Others
What kind of water do you drink mainly?			8	2	
What kind of water do you use for cooking mainly?	5		4	1	
What kind of water do you use when you wash hand after lavatory mainly?	7		3		
What kind of water do you use when you wash hand before eating meal mainly?	5		4	1	
What kind of water do you use when you wash body mainly?	7		3		

Figure 1: Container for rainwater storage

Table 2: Washing material (Number of subjects)

Questions	Nothing	Soap	Liquid soap	Others
What kind of washing material do you use when you wash hand after lavatory mainly?	5	5		
What kind of washing material do you use when you wash hand before eating meal mainly?	5	5		
What kind of washing material do you use when you wash body mainly?	2	8		

Table 3: Subjects' knowledge of harmful substances in water

Questions	Answers			
Do you know the word "ARSENIC"?	YES		NO	
	10			
If "YES" Do you know the effects of ARSENIC on human health?	YVW	YES	NO	NAA
	1	6	3	
Do you know bacteria? YES NO	YES		NO	
	5		5	
If "YES" Do you know the effects of bacteria in drinking water on human health?	YVW	YES	NO	NAA
		4	1	
Do you know salt? YES NO	YES		NO	
	10			
If "YES" Do you know that drinking saline water every day can be harmful to human?	YVW	YES	NO	NAA
	5	3	2	

YVW: Yes very well, NAA: Not at all

Table 4: Subjects' attitude to water safety

Questions	Answers			
Are you anxious about safety of pond water?	VA	A	NA	NAAA
	6	1	2	1
Are you anxious about safety of Tube-well water?	VA	A	NA	NAAA
	5	2	2	1
Are you anxious about safety of stored rainwater?	VA	A	NA	NAAA
		6	3	1
Are you anxious about drinking saline water?	VA	A	NA	NAAA
	5	2	2	1
Do you have any anxiety that children drink pond water?	VA	A	NA	NAAA
	5	2	1	2
Do you have any anxiety that children drink Tube-well water?	VA	A	NA	NAAA
	5	1	3	1
Do you have any anxiety that children drink stored rainwater?	VA	A	NA	NAAA
	5	1	3	1
Do you have any anxiety that children drink saline water?	VA	A	NA	NAAA
	5	2	2	1
Do you think drinking water without arsenic is necessary for your better life?	YVW	YES	NO	NAA
	2	8		
Do you think drinking water without bacteria is necessary for your better life? ※	YVW	YES	NO	NAA
		6		3
Do you think drinking water without saline is necessary for your better life?	YVW	YES	NO	NAA
		10		

※ One subject did not answer the question about bacteria.
VA: Very anxious, A: Anxious, NA: Not anxious, NAAA: Not anxious at all
YVW: Yes very well, NAA: Not at all

4. DISCUSSION

It is considered that many residents use pond water and/or ground water as living water in rural areas of Bangladesh. The pond water is supposed to contain pathogenic bacteria and salt, especially in the southern part of Bangladesh. In the present study, half of subjects in Parbayarjhapa used pond water for cooking. As pond water is thought to be boiled, the water is sterilized and bacteria may be killed and rather decreased. However, salt is not decreased but condensed by boiling. Because salt is related to various diseases such as stomach cancer

[Tsugane, 2004], heart disease [Tuomilehto, 2001], stroke [Nagata, 2004], diabetes mellitus [Feldstein, 2002] and so on, use of pond water must be harmful to health. Chloride ion concentration of pond water and ground water in Parbayarjhapa were actually measured in July of 2012. The concentration of pond water was 6750 mg/L and the concentration of ground water was 440 mg/L (WHO's Guidelines for Drinking-water Quality: 250 mg/L). The high concentrations even in the rainy season indicate that the concentrations are much higher in the dry season.

Various bacteria exist basically in a pond. Then most of subjects washed body in a pond. In addition, a pit toilet is located near a pond. That is why, pond water might be contaminated. However some washed hand with pond water and without any soap before meal. This way is a very risky manner. Tube-well water and soap should be used for infectious prevention at least before meal. Otherwise bacteria invade a human body through a hand and a mouth at meal. There was only one tube-well in the village. It is believed that shortage of tube-well leads to using pond water. Furthermore, there was no PSF in Parbayarjhapa. Though effects of PSF on health are unknown, nevertheless, the condition of having no PSF must be improved. Considering the life style and the pond condition, quality and usage of living water in Parbayarjhapa are not desirable from the viewpoint of human health.

It is difficult to get international financial support for water. Even if advanced apparatus is established in Parbayarjhapa, it would be not easy to maintain it. Breakdown of the apparatus could really happen due to a flood or a cyclone. Talking such local conditions into consideration, a small household-oriented apparatus is suitable in Parbayarjhapa. Small apparatus of a required number could be suitabler and more realistic than a large equipment in terms of effective utilization of complicated and small land. TSS is recommended as such a small household-oriented apparatus. The concept of TSS is simply evaporation by solar energy [Islam, 2004]. TSS is comprised of a tubular cover and a black-colored semi-circular trough. The tubular cover is made of a curled transparent vinyl chloride sheet. The size of the trough and the vinyl chloride sheet can be changed according to condition and/or objective volume of water. When water containing various substances in the trough is heated by solar light, only water is evaporated and separated from the substances. Drops of evaporated water adhered to cover sheet, then drops fall along down to the vessel and are collected. Because composition of TSS is simple, the method for making and maintenance is not so complicated. The materials are affordable.

A preliminary experiment was carried out in order to check the ability to remove or reduce several harmful substances (Data not shown in RESULTS). At first, TSS was able to remove completely Escherichia coli (E. coli), which was a kind of bacteria, from primary treated water of sewage. The water distilled by TSS had contained no E. coli. Second, TSS reduced arsenic in the solution to below detection limit (0.005 mg/L). Before application, the solution had contained arsenic at the concentration of 0.09 mg/L. Although arsenic was experimentally added to pure water and adjusted to the concentration of 0.1 mg/L, the actual measured value was slightly low after adjustment. Since the condition was about 10 times of WHO's standard (0.01 mg/L) and rather intensive. Even so, TSS removed arsenic. Lastly, TSS removed also salt from salt water. The concentration was adjusted to 2.8%, which was nearly identical to saline water. It is supposed that removal of E.coli was caused by the sterilizing effect of ultra violet and heat, while arsenic and salt did not evaporate and remained as a residue due to the difference of steam pressure. Because not only size of TSS is changeable depending on feature and area of land, but also TSS remove various harmful substances, TSS is expected as a resolution of water problem in Parbayarjhapa.

In the present study, it became clear that most of all subjects had diarrhoea and the frequency was serious. Unsanitary condition of water usage in Parbayarjhapa is considered to cause diarrheal diseases. Diarrhoeal disease is a major causes of child death in Bangladesh. Hence the condition might be unsanitary not in Parbayarjhapa but also whole Bangladesh. The condition should be improved immediately.

All subjects knew salt and arsenic and most of them had understood those harmful effects on human health. Among 7 subjects, who knew arsenic was harmful, 4 subjects knew skin problem as a specific effect of arsenic. However among 8 subjects, who knew salt was harmful, nobody knew any specific harmful effect of salt (Data not shown in RESULTS). As for bacteria, those who knew bacteria was only half of subjects (Table 3). Actually, the result was reflected in the fact that the sense of crisis was somewhat less and 3 subjects were not aware of the risk (Table 4). The view might lead to diarrhoea. Risk of arsenic had pervaded Parbayarjhapa, while their knowledge was inclined toward arsenic. As various knowledge about water is required for better healthy life, also education other than arsenic must be provided much more.

As for pond water and tube-well water, 7 subjects were anxious about the safety, as for stored rainwater, 6 subjects were worried about the safety. Among these three different kinds of water, rain water may be safer because neither salt nor arsenic is contained in rain water. However, as Figure 1 suggests, there is possibility that rain water is contaminated by bacteria in the process of collection and storage; water needs to be boiled and sterilized before use.

Hearing survey directly from the residents was also executed. Some residents stated that those engaged in prawn culture, especially female workers, had inflammation of reproductive organ. Although the quality of culture ponds was not investigated, judging from the appearance, the water was muddy and the level of suspended solids seemed high. To the best of knowledge, there are few reports about inflammation and dermatitis caused by salt. Therefore, a kind of bacteria living in unhygienic water is supposed to cause the disease.

When hearing from some residents without the questionnaire was conducted, the tendency was mostly the same as other than the subjects. That is why, although the number of subjects was small, the overall tendency might be reflected in the result of present study.

5. CONCLUSIONS

Quality and usage of living water in Parbayarjhapa were not desirable from the viewpoint of human health. Any solution to water problem like TSS should be introduced. Moreover, because knowledge of water of residents may incline to arsenic, education concerning other than arsenic, such as salt and bacteria, is necessary for better health.

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REFERENCES

Feldstein C.A. (2002) “Salt intake, hypertension and diabetes mellitus.” Journal of Human Hypertension, Vol. 16(Suppl 1), S48-S51.

Islam, K.M.S., Fukuhara, T., Asano, F. (2004). “Mass Transfer in Tubular Solar Still.” Proceedings of 59th Annual Meeting, Japan Society of Civil Engineering, Section 7-236, 471-472.

Nagata, C., Takatsuka, N., Shimizu, N., Shimizu, H. (2004). “Sodium intake and risk of death from stroke in Japanese men and women.” Stroke, Vol. 35(7), 1543-1547.

Stauber, C.E., Kominek, B., Liang, K.R., Osman, M.K., Sobsey, M.D. (2012). “Evaluation of the impact of the plastic BioSand filter on health and drinking water quality in rural Tamale, Ghana.” International Journal of Environmental Research and Public Health, Vol. 24(9), 3806-3823.

Tsugane, S., Sasazuki, S., Kobayashi, M., Sasaki, S. (2004). “Salt and salted food intake and subsequent risk of gastric cancer among middle-aged Japanese men and women.” British Journal of Cancer, Vol. 90(1), 128-134.

Tuomilehto, J., Jousilahti, P., Rastenyte, D., Moltchanov, V., Tanskanen, A., Pietinen, P., Nissinen, A. (2001). “Urinary sodium excretion and cardiovascular mortality in Finland: a prospective study.” Lancet, Vol. 357(9259), 848-851.

World Health Organization (2013). World Health Statistics 2013, http://www.who.int/gho/publications/en/.

A NEW CLIMATE THREAT FROM SLCPS AND THEIR FAST-ACTION MITIGATION MEASURES IN CONTEXT OF BANGLADESH

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ABSTRACT

Short-lived climate pollutants (SLCPs) which include black carbon, tropospheric ozone, methane and some hydroflurocarbons (HFCs); have relatively short lifetime in atmosphere - a few days to a few decades. These are the most important contributors to the prolongation of global warming after carbon-dioxide (CO_2); which persist in the atmosphere for millenniums. These SLCPs are also dangerous air pollutants, with various detrimental impacts on human health, agriculture and ecosystems. And this pollution is an up burning issue for Bangladesh, as a developing country. This study explored the sources of SLCPs in context of Bangladesh and effects of these pollutants on our local environment. The assessment of present scenario and prediction of future with the present data are done using LEAP Model; energy analysis software. The result from LEAP Model shows that the emission of these short lived pollutants is going to threat human health and our productivity of agricultural goods; while our country's economy totally depends on agriculture. A short guide-line to induce mitigation measures for reducing these SLCPs is suggested at the end for better future.

Keywords: Environmental refugee, global warming, lifetime, crop yields, implementation

1. INTRODUCTION

Climate change is no longer something to happen in future, it is here and now. Bangladesh is among the countries that are expected to be worst affected by climate. Since this country achieved independence in 1971, GDP has reached to 6.8% by 2012; the population growth rate has declined from around 2.9% per annum in 1974 to 1.3% in 2011 and the country is now largely to food secure. Over the last 20 years, growth has accelerated and the country is on the track to become a middle income country by 2021 when it celebrates its 50 years of independence. Also Bangladesh Government is on the right track of reaffirming its commitment to the MDG targets, including halving poverty and hunger by the year 2015 through a strategy of pro-poor growth and climate resilient development. But climate change is severely throwing challenge to the country's ability to achieve the high rate of economic growth needed to sustain these reductions in poverty. In coming years, it is predicted that there will be increasingly frequent and severe floods, tropical cyclones, storm surges and droughts, which will disrupts the life of nation and economy. In the worst case scenario, unless exiting coastal polders are strengthen and now ones built, sea level rise could results in displacement of millions of people – 'environmental refugees' - from coastal regions and have huge adverse impacts on the livelihood long term health of large proportion of the population [1]. It is essential that Bangladesh prepares now to face the challenge ahead and safeguard the future economic wellbeing and livelihood of her people. The figure 1 shows the vulnerable areas to different types of natural calamities of Bangladesh.

2. SHORT LIVED CLIMATE POLLUTANTS:

According to scientists Ramanathan and Feng [2], human emission of greenhouse gases as of 2005 may have already committed the planet to 2.4°C of warming, pushing the climate system dangerously close to the boundaries of dangerous anthropogenic interference and predicted climate tipping points. Climate tipping points are predicted changes in the earth's climate systems which have the potential to trigger abrupt, irreversible and catastrophic shifts that can overwhelm ecosystem's and society's ability to adapt. A few examples of predicted

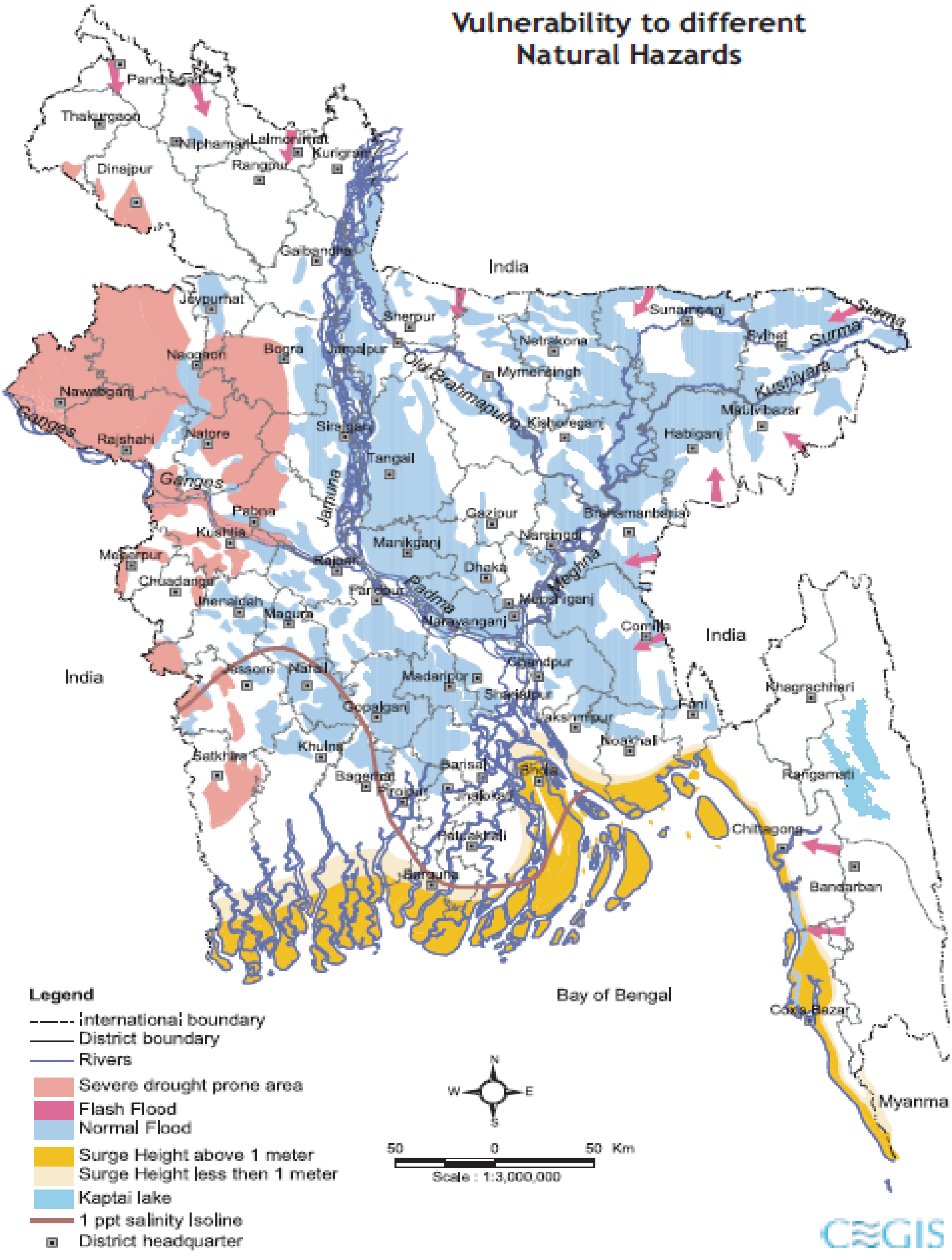

Figure1: Areas of Bangladesh affected by different types of climate related disasters (Source: Centre for Environmental and Geographical Information Services, Dhaka)

tipping points include the melting of Arctic permafrost and sea ice (Figure: 2), melting of the Greenland ice sheet, dieback of the Amazon rainforest, disappearance of the Hindu-Kush-Himalayan-Tibetan glaciers, and the shutdown of the Atlantic Thermohaline Circulation.

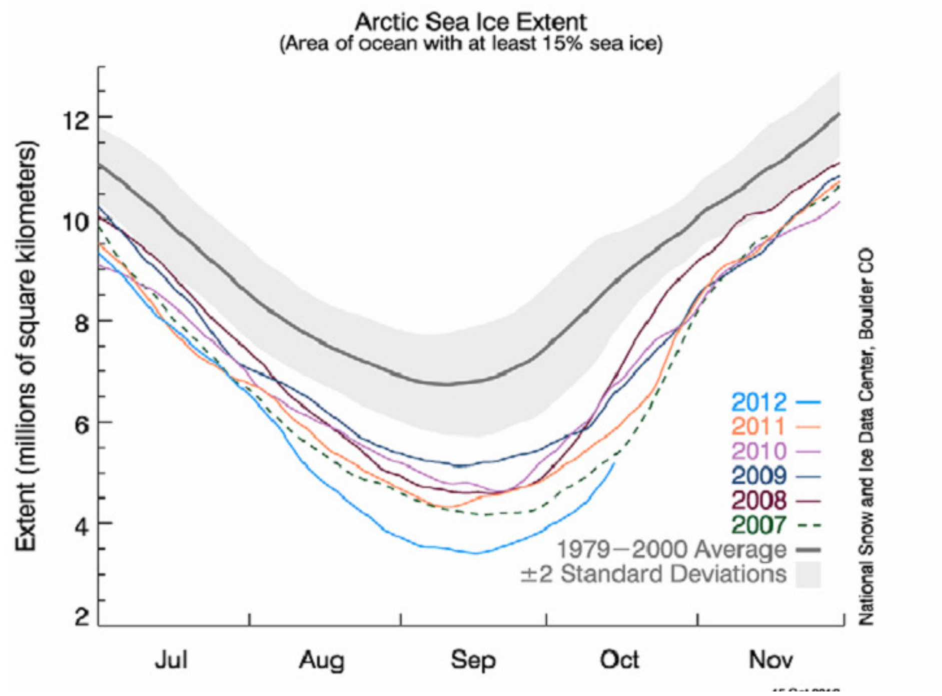

Figure 2: The graph above shows Arctic sea ice extent as of October 15, 2012, along with daily ice extent data for the previous five years. 2012 is shown in blue, 2011 in orange, 2010 in pink, 2009 in navy, 2008 in purple, and 2007 in green. The gray area around the average line shows the two standard deviation range of the data. (Source: National Snow and Ice Data Centre)

International action to reduce CO_2 emissions, while necessary to manage long-term warming, will do nothing to prevent already accelerating harms in critical vulnerable regions. It is only by cutting the powerful non- CO_2 short-lived climate pollutants (SLCPs) refe to four pollutants: black carbon, tropospheric ozone, methane and hydro-fluorocarbons (HFCs) that we can produce meaningful and immediate mitigation and prevent reaching temperature thresholds for predicted climate tipping points in the next few decades. These short-lived climate pollutants, which are estimated to account for as much as 40-50% of positive anthropogenic radiative forcing, have atmospheric lifetimes of weeks to decades. Cutting their emission decreases their effect on the global radiative budget to zero on a short time scale and pushes back the threshold for predicted tipping points for decades.

In addition to their contribution to climate change and passing thresholds for climate tipping points, emissions of non-CO_2 short-lived climate pollutants have significant adverse effects on:

- Health (Due to black carbon, also ozone)
- Agriculture (crop yield; due to methane)
- Ecosystem productivity (Due to ozone)

Their current impacts and predicted future impacts of these emissions are so severe that failure to take mitigating actions should be recognized as a violation of fundamental human rights.

3. METHODOLOGY

The methodology followed for studying the features and effects of these short lived climate pollutants is summarized here. First of all, we identified the major sources of these pollutants and collected the emission data based on available information. Then, we reviewed current status of air pollution and emission inventories. LEAP, the Long range Energy Alternatives Planning System, developed at the Stockholm Environment Institute, is a widely-used software tool for energy policy analysis and climate change mitigation assessment. This software is used in context of Bangladesh to predict the upcoming SLCPs emission in near future. Then abatement measures of these pollutants are suggested for developing National Action Plan.

4. SOURCES OF SHORT LIVED CLIMATE POLLUTANTS

United Nation Environment Programme has identified the major sources of SLCPs on a global scale, including dominant sources in different regions of the world. But, no systematic work has so far been carried out for assessment of SLCP sources in Bangladesh. However, a number of Government reports/ communications and scientific studies are available on emissions and GHG inventory developed by Department of Environment of Bangladesh (DoE) under different projects like CASE projects, Malé declaration, etc. This Section provides an overview of the major sources of SLCPs in Bangladesh:

4.1 Black Carbon

Black carbon is a major component of soot and is produced by incomplete combustion of fossil fuel and biomass. It is emitted from various sources including diesel cars and trucks, residential stoves, forest fires, agricultural open burning and some industrial facilities. It has a warming impact on climate 460-1500 times stronger than CO_2. Its lifetime varies from a few days to a few weeks. When deposited on ice and snow, Black Carbon causes both atmospheric warming and an increase of melting rate. It also influences cloud formation and impacts regional circulation and rainfall patterns. In addition, black carbon impacts human health. It is a primary component of particulate matter in air pollution that is the major environmental cause of premature death globally. Approximately 3 billion people are subject to elevated exposure in black carbon hotspots in East and South Asia, Southern Africa, and the Amazon Basin. In Africa alone, where biomass sometimes accounts for 90% of household fuel supply, studies suggest that indoor air pollution will cause approximately 9.8 million deaths by the year 2030.

The major sources of Black Carbon include the transportation sector (e.g., diesel vehicles), residential cooking (traditional stoves) and heating, industrial sector (e.g., brick kilns, rice parboiling) and agricultural burning; figure 3. There are about 5,000 brick kilns in Bangladesh, producing over 10 billion bricks per year [3]. Among the kilns, majority (about 92%) are fixed chimney kilns (FCKs), while the rest are zigzag brick kilns (ZKs), hybrid Hoffman brick kilns (HHKs) and other technologies. Almost all the brick kilns use coal as the primary fuel; however, wood is still used in many kilns. Most kilns operate during dry season, aggravating the already poor air quality.

Traditional rice parboiling units operating in the country have also been identified as a major source of Black Carbon. There are over 17,000 traditional parboiling mills and over 450 automatic parboiling mills operating over the country, producing about 28 million tons of rice annually [3]. The traditional parboiling mills typically run at low thermal efficiency (15-30%), using rice-husk as fuel, which are major sources Black Carbon emission. Commonly women (often accompanied by their children) work in subsequent drying and processing of parboiled rice, thus exposing them to severe air pollution.

Figure 3: Different sources of Black Carbon; Rice Parboiling [Left], Brick Kilns [Middle] & Diesel Engine [Right]

Vehicular emissions are the second largest contributor to air pollution after brick kilns. High sulphur containing diesel engines are used for passengers and fright movement. Almost 85% of total trucks and 35% of buses are diesel based which are the major sources of Black Carbon emission [4].

Like in many developing countries, biomass is extensively used in rural areas of Bangladesh, primarily for cooking. An estimated 80 million tons of biomass fuel is used at approximately 30 million households and 1 million institutions use traditional cook-stove (with 5-15% thermal efficiency) annually for cooking, producing huge quantities of Black Carbon and other pollutants [5].

The contribution of power plants to air pollution/ emission (PM/ BC, other pollutants) in Bangladesh has not been large. But emission from the numerous diesel generators that supplement the grid electricity supply during load-shedding is another concern. These generators emit much closer to the people, with potentially large health impacts; however, no data is available on emissions from these generators.

The LEAP model (Version: 2012.0.0.37) was utilized to compile Black Carbon emissions for the years 2005-2030 for Bangladesh, in addition to identifying the sectors and activities making up these emissions, figure 4.

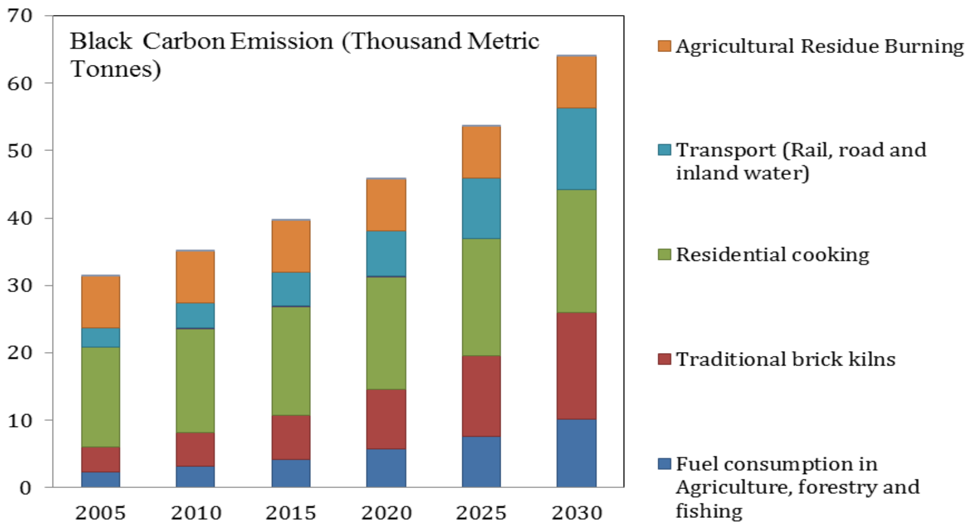

Figure 4: Estimation of Black Carbon emissions from different sectors (in thousand Metric Tons per year) for the reference years 2005 – 2030.

4.2 Methane

Methane (CH_4) is a greenhouse gas that is over 20 times more potent than CO_2, and has an atmospheric lifetime of about 12 years. It is produced through natural processes (i.e. the decomposition of plant and animal waste), but is also emitted from many man-made sources, including coal mines, natural gas and oil systems, and landfills. Methane directly influences the climate system and also has indirect impacts on human health and ecosystems, in particular through its role as a precursor of tropospheric ozone.

According to UNFCC, the major sources of methane emissions include domestic wastewater, livestock, paddy, poultry, and solid waste in Bangladesh. According to the second national communication of Bangladesh, estimated methane emissions from these sources for the year 2004-05 are 621.08 Gg, 493.16 Gg, 380.75 Gg, 84.79 Gg, and 16.50 Gg respectively [6].

Among all the cities in Bangladesh, only Dhaka has one operational municipal sewage treatment plant (Pagla sewage treatment plant) that serves only a small fraction of its population. Wastewater and septic tank overflows usually find their ways into low-lying areas, lakes, khals, and rivers within and surrounding the urban centers. Apart from causing sever environmental pollution; wastewater and sludge accumulated in these water bodies undergo anaerobic decomposition producing methane.

Currently there are over 65,000 poultry farms and over 59,000 livestock (cattle, goat, buffalo and sheep) farms in the country [3]. Together they are charged for 36% of total methane emission in this country. Anaerobic decomposition of organic material in flooded rice fields produce methane, which escapes to atmosphere. In this country, almost 23.8% of total methane is emitted from 12000 thousand hectare land of rice cultivation.

Coal mining and leakage in gas production and from gas transmission lines are other sources of methane. Among the discovered coal fields in Bangladesh, only Barapukuria is being developed employing underground method of mining. Islam and Hayashi estimated the coal bed methane concentration for coals at Barapukuria to be 6.51-12.68 m^3/t [7]. But coal bed methane extraction and recovery of methane (CMM) from ventilation air are not practiced at the Barapukuria coal mine. There is about 2100 km of natural gas transmission pipeline (pressure 1000 psig), maintained by four gas transmission and distribution companies (TGTDCL, GTCL, BGSL, JGTDSL) in this country. The total length of distribution line is much higher. There is no specific data on leakage of gas from transmission and distribution system.

The LEAP model also reveals that methane emissions for Bangladesh have been increasing each year (around 50 Metric ton/year) in agriculture sector, while emissions from other sector have been steady for the years 2005-2030, figure 5.

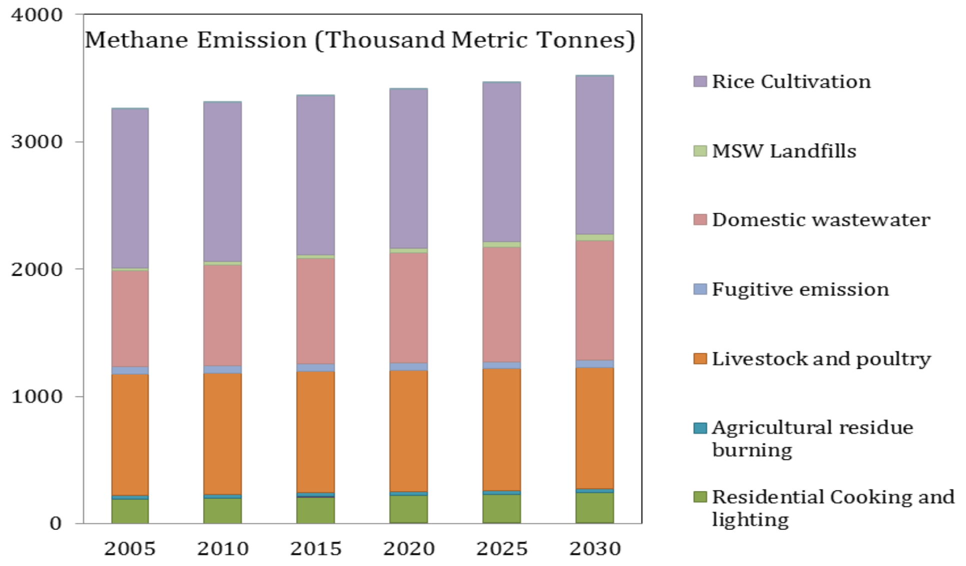

Figure 5: Estimation of Methane emissions from different sectors (in thousand Metric Tons per year) for the reference years 2005 – 2030

4.3 Hydro-fluorocarbons (HFCs)

The current contribution of HFCs to direct climate forcing is calculated to be approximately 0.012 Wm-2. However, the annual increase in HFC forcing in past few years is significant when compared to that of other substances. During a five year periods, from mid 2003 to mid 2008, total climate forcing from HFCs increased by about 0.006 Wm-2. With this increasing rate, it is predicted that in 2050 HFCs climate forcing is going to replace that of CFCs which is phased out in recent years.

HFCs have been introduced into commercial use largely because they have proven to be effective substitutes for ODSs. Sources are where HFC used, refrigeration and air-conditioning equipment in home, other buildings and industrial operations (600 million tons in CO_{2eq}, ~55% of total HFC use in 2010) and for air-conditioning in vehicles (250 million tons in CO_{2eq}, ~24%). Smaller amounts are used for foam products (~11%), aerosols (~5%), fire protection system (~4%) and solvents (~1%). The use of HFCs is increasing rapidly as result of global economic development and population growth at 8-9% per year which is notably greater than the recent increases of about 4% per year in case of CO_2 and about 0.5% per year in case of methane [8].

4.4 Tropospheric Ozone

Tropospheric or ground level ozone (O_3) is the ozone present in the lowest portion of the atmosphere (up to 10-15km above the ground). It is responsible for a large part of the human enhancement of the global greenhouse effect and has a lifetime of a few days to a few weeks. It is not directly emitted but formed by sunlight-driven oxidation of other agents, called ozone precursors, in particular methane (CH_4) but also carbon monoxide (CO), non-methane volatile organic compounds (NMVOCs) and nitrogen oxides (NOX). Tropospheric ozone is a harmful pollutant that has detrimental impacts on human health and plants and is responsible for important reduction in crop yields.

As tropospheric ozone is a secondary pollutant, there are many sources which cause formation of this ozone. Methane is the one which can generate ozone after emitted into atmosphere. HFCs also cause the formation of this ozone. So, the sources of methane and HFCs are potential sources of ozone also.

5. EFFECTS OF SHORT LIVED CLIMATE POLLUTANTS

It is widely accepted that particulate matter (PM) is the major pollutant of significant health concern internationally and in Bangladesh. In recent times, the adverse effects of Black Carbon, a major component of PM, has attracted much attention. Numerous epidemiological and toxicological studies related elevated Black Carbon concentration with an increased risk of premature mortality. Toxicological studies suggest that Black Carbon may not be a major toxic component of fine PM, but it may operate as a universal carrier of a wide variety of chemicals of varying toxicity to the lungs, the body's major defense cell and possibly the systemic blood circulation (WHO, 2012).

Apart from direct health effects, Black Carbon causes warming of the atmosphere by a number of different processes. When deposited on ice and snow, Black Carbon reduces the albedo of these surfaces, increasing both atmospheric warming and melting rate caused by increased absorption of heat by the darker snow and ice (UNEP, 2011). Black Carbon aerosols have a large impact on regional circulation and rainfall patterns (e.g., monsoon) as they cause significant asymmetry in heating patterns over a region; the impact of Black Carbon on regional weather patterns and regional warming is more certain than its impact on global warming (UNEP, 2011).

In the troposphere, O_3 is an important greenhouse gas. It also affects the yield of crops and has adverse impacts on diversity and growth of plant communities (UNEP, 2011; UNEP, 2012). Being a powerful oxidizing gas, it also affects human health, e.g., by causing oxidative stress in lungs (UNEP, 2011). According to UNEP (2011), on a global scale, reducing Black Carbon and methane has the potential given below [9]:

- To prevent over 2 million premature deaths each year from indoor and outdoor air pollution (of which an estimated 159,000 in Bangladesh);
- To avoid annual crop loss of over 30 million tons (of which over 20 million tons in Asia and the Pacific regions); and
- To slow down the warming expected by 2050 by as much as 0.5 °C;

6. ABATEMENT MEASURES

The assessment for determining the abatement measures of SLCPs was made through detailed analysis of existing and planned (draft) policies, rules and regulations; ongoing and future programs/ projects in different sectors; through consultation with stakeholders at two workshops held on 27 February and 19 March 2013, and through one to one interview of key stakeholders. The measures are given below according to pollutants:

Measures for black carbon:

- Elimination of high emitting vehicles in road transport
- Encourage diesel to CNG switch through incentives
- Promoting cleaner diesel
- Introduction of clean burning biomass stoves for cooking
- Substitution of biomass cook stoves with stoves using clean-burning fuel
- Replacing traditional brick kilns with modern technologies
- Introduction of improved rice parboiling technology

Measures for methane:

- Promote recovery and utilization of methane from coal mines
- Recovery and utilization of vented associated gas, and improved control of unintended fugitive emission/ leakage from natural gas production and processing
- Reduction of leakage from natural gas transmission and distribution system
- Promoting septic sludge and fecal sludge collection, treatment and recycling
- Establishment/ expansion of sewerage system and municipal wastewater treatment plant in major urban centers
- Separation and treatment (composting, anaerobic digestion, refuse-derived fuel) of biodegradable municipal waste
- Landfill methane gas collection with combustion or utilization
- Intermittent aeration (AWD) of continuously flooded rice paddies
- Control of methane emission from livestock, through anaerobic digestion of manure from cattle and poultry

Measures for HFCs:

- Avoid the phase in HFCs as effort of phase out of HCFCs
- Replace the existing HFCs based equipment
- Recovery and recycling of HFCs
- Promote the destruction of HFCs

7. CONCLUSIONS

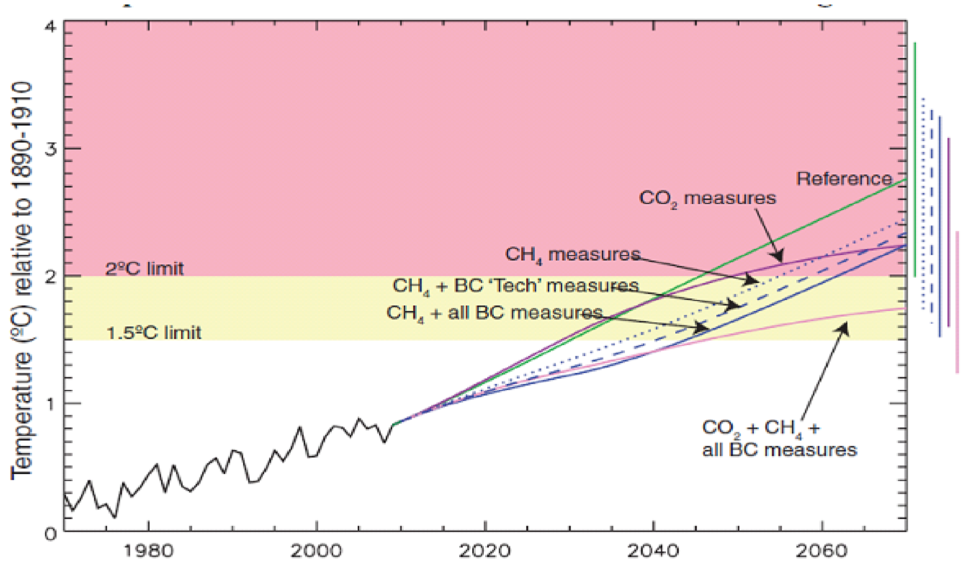

Figure 6: Temperature Rise Predictions under Various Mitigation Scenarios (Source: Shindell *et al.*, SCI 2012.)

Reducing CO_2 remains the top priority, but we also need to simultaneously reduce SLCPs in order to achieve near-term benefits that will keep us from losing the climate battle while massive. Longer term CO_2 emissions reductions are carried out. We also need to perfect and implement strategies to deliberately reduce excess CO_2 from the atmosphere on a time scale of decades. It is established that, the combination of Methane and Black Carbon measures along with substantial CO_2 emissions reductions [under a 450 parts per million (ppm) scenario] has a high probability of limiting global mean warming to <2°C during the next 60 years, something that neither set of emissions reductions achieves on its own (shown in figure 6). So, the take-away message from the science and the growing impacts is the need for speed and the importance of fast-action mitigation to address all causes of climate change.

REFERENCES

[1]MoEF (2009), "Bangladesh Climate Change Strategy and Action Plan 2009", Ministry of Environment and Forests, Government of People's Republic of Bangladesh, xviii+ 76pp
[2]Ramanathan V. and Feng Y., "On Avoiding Dangerous Anthropogenic Interference with the Climate System - Formidable Challenges Ahead", National Academy of Science of USA, September, 2008
[3]Bangladesh Bureau of Statistics, The statistical yearbook of Bangladesh 2011, Ministry of Planning, Bangladesh Government, 2011
[4]DoE (2012a), "Revisions of Vehicular Emission Standards for Bangladesh, Draft Final Report – Part 1", CASE Project, Department of Environment, December 2012
[5]Asaduzzaman M., Barnes D. F. and Khandakar S. R., "Restoring balance: Bangladesh's rural energy realities, Energy Sector Management Assistance Program", Development Economics Research Group, 2011
[6]GoB/ UNDP (2012), "The Second National Communication of Bangladesh to the UNFCC", Ministry of Environment and Forest, Government of Bangladesh, and the United Nations Development Programme
[7] Islam and Hayashi, "Challenges of Coal-bed Natural Gas Development in Northwestern Bangladesh", Search and Discovery Article, February 2011
[8]UNEP (2011), "HFCs: A Critical Link in Protecting Climate and the Ozone Layer", United Nations Environmental Programme, November 2011
[9]WHO (2012), "Health Effects of Short Lived Climate Pollutants", World Health Organization Regional Office for Europe, September 2012

STUDY ON WATER PURIFICATION BY SOLAR ENERGY

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ABSTRACT

The principle focus of this thesis project is using solar energy to purify water so that people can easily achieve safe drinking water. To achieve this goal, a solar water purifier is constructed consists of an air tight box type solar collector and a ceramic filter. It is constructed for one family with four members which can contain 17.5 liter water per day. For finding out the performance of the solar collector several tests are run in summer, winter and rainy seasons by means of bacteria culture and its removal. The solar collector performs appreciably by raising the temperature of water which is enough to disinfect water in every season. The effectiveness of solar disinfection system for pond water contaminated with microorganisms is also evaluated so that it can be used easily for household drinking purpose. It is observed that within 8 hours, Total coliform and E. coliform are completely removed from pond water. The other quality parameters present in pond water such as water temperature, pH, turbidity and color are also measured with time. It is seen that these parameters remain within the allowable limit after using the solar water purifier. Considering all the outputs, it can be said that the solar water purifier can be of great use to the people in Bangladesh.

Key words: Box Type Solar Collector, Bacteria Culture, Solar Disinfection, Pond Water Parameters, Additional Filter Design.

1. INTRODUCTION

A shortage of fresh water is very crucial problem that is continuously increasing day by day, due to population growth and changes in weather condition. According to the latest estimates of the WHO/UNICEF Joint Monitoring Program for Water Supply and Sanitation (JMP), released in early 2013, 768 million people still use unsafe drinking water sources. Every 8 seconds, a child is being died from water related disease around the globe. 50% of the people in developing countries suffer from one or more water-related diseases. 80% of diseases in the developing countries are caused by contaminated water (Jagadish, 1998). Half of the world's hospital beds are filled with people suffering from a water-related disease (UNDP, 2006). The UN’s fourth World Water Development Report recommends much broader collaborative and integrative water management approaches and innovations will be needed to avoid future conflicts over water among nations and within nation. In the rural areas of most developing countries the condition of drinking water is being very worse. In the most of the region of our country the water is being contaminated by arsenic. Now it has become very severe problem in our country. Also continuously use of underground water causes the underground water level to be decreased day by day. In southern part and other coastal area of our country are lacking of fresh drinking water. Rain water is being collected in small pond in those areas. Without any kind of treatment water is being drunk by people living in those areas, which is very unhygienic.Conventional technologies are being used to disinfect water are: ionization, chlorination and artificial ultra violet radiation. These technologies require sophisticated equipment, huge capital investment and require skilled operators. Boiling of water requires about 1 kg of wood/liter which results in deforestation in developing countries. Also halogen or calcium hypochlorite tablets or solutions (sodium hypochlorite at 1 to 2 drops per liter) are used to disinfect drinking water. These methods are environmentally unsound or hygienically unsafe when performed by a layperson. Misuse of sodium hypochlorite solution poses a safety hazard (Jagadish, 1998). Rainwater collection, pond sand filter (PSF), pond water for drinking and other domestic purposes is practiced by individuals is almost every country in the world. Studies reported in the literature reveal the prevalence of microbial and other contaminants in roof collected rainwater (Meera and Ahmed, 2006). This indicates the need for proper treatment of rainwater of water if it is used for drinking. Low cost, simple and easy to use household treatment methods are suitable for treating water for rural communities. Solar disinfection are some of the systems developed for treating water at household level. Treatment to control waterborne microbial contaminants by exposure to sunlight in clear vessels that allows the combined germicidal effects of both ultra violet radiation and heat has been developed and both effects contribute to the inactivation of water borne microbes. It can be seen that it is not required to boil the water in order to kill 99.9% of the microorganisms. Heating up water to 60 -65°C for one hour has the same effects (Walker, 2004). The objective of the present study was to assess the suitability of solar disinfection for household treatment of water in Bangladesh. The removal kinetics of microorganisms of Bangladesh was studied. Moreover, coliform organisms' inactivation kinetics for bottles of different sizes and constituting material and the effect of turbidity of water, pH of water and presence of ultraviolet ray in the sunlight etc. on bacterial inactivation rate were also studied.

2. METHODOLOGY

2.1 Research Strategy of Methodology

The various steps followed to complete the thesis project are briefly represented in a block diagram in figure 3.1.

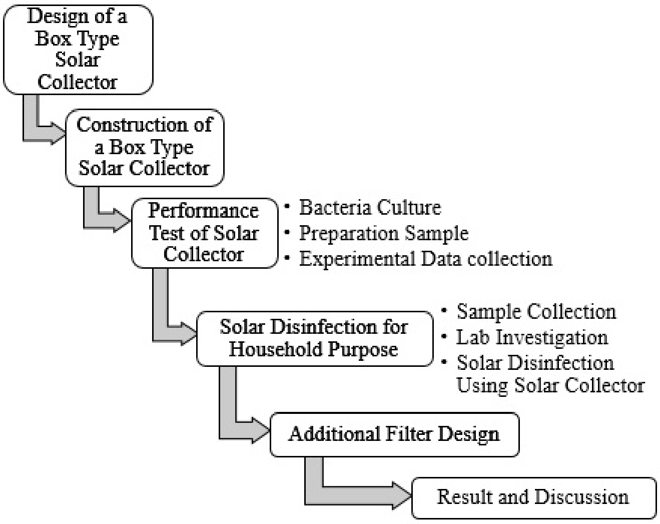

Figure 18: Research Strategy of Methodology

2.2 Design and Construction of a Box Type Solar Collector

The box type solar collector was mainly designed for removal of bacteria from the waste surface water such as pond water. As the bacteria are completely removed at 60-65°C, so the main consideration of the design was to increase temperature of water inside solar collector box up to 60-65°C. That's why at first a container was designed of dimension 12´´ X 17.5´´ which can contain 17.5 liter so that it can meet the demand of a family of four members for a day easily. The inside surface of the container was coated with black color so it can absorb maximum heat energy. During the selection of color, it was kept in mind that the color should not affect the water adversely. Also the top surface of the container was tilted about 10 degree with respect to horizontal so that it can receive maximum heat energy over the day. A rectangular wooden frame with dimensions of 15.5´´ X 21.5´´ was designed to hold the container and the total system. Following these design considerations, a box type solar collector was constructed as shown in figure 2. Carpus cotton was used to insulate the clearance between the container and wooden frame. Again, the top surface was covered with a 6 mm thick glass to create a greenhouse effect.

As pond water is not only contaminated by bacteria, it has other impurities such as pH, color, turbidity etc. So for purifying waste water, a filter design must be taken under consideration. That's why, a filter was constructed. By mixing the soil (passed through 30 no. sieve), rice husk (passed through 40 no. sieve) and water in a ratio of soil and rice husk of 80:20, the mixture was dried in the sun. After that the dried mixture was burnt. The constructed filter was added to the outlet of solar collector box to complete the construction of solar water purifier as shown in figure 3.

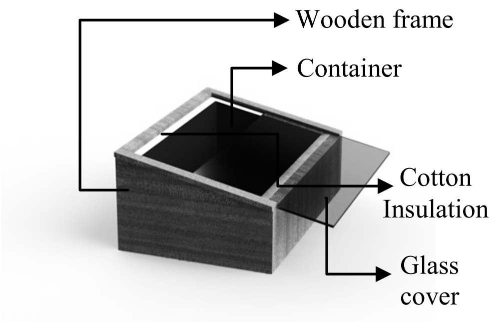

Figure 2: Design of a box type solar collector

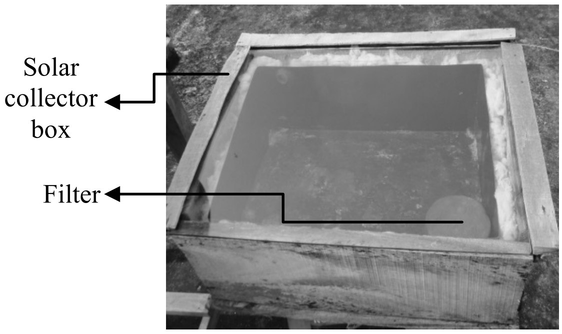

Figure 3: Addition of filter to solar collector box

2.3 Performance Test of Box Type Solar Collector

Through bacteria culture, fifteen waste water samples were prepared in ranges between 50 - 5000 numbers of total coliform in order to test the performance of the solar collector. Thus it was found how much bacteria can be removed by solar energy.

2.3.1 Bacteria Culture and Sample Preparation

In order to culture bacteria, coconut mesh was sized about one inch so that fermentation can occur properly. Then drain water and tape water were collected as shown in figure 4 and along with coconut mesh were kept together in a bucket to culture bacteria as shown in figure 5.

Figure 4: Collection of drain water

Figure 5: Fermentation process

Figure 6: Sample preparation

Figure 7: Experimental Setup

After ten days, water from mixture of drain water, coconut mesh and tape water was collected and number of Total coliform in it was determined. Then fifteen samples were prepared as shown in figure 6.

2.3.2 Experimental Data Collection

In dry season, for 8 hours, from 8AM to 4PM, prepared samples were placed in the box type solar collector. After that, total coliform tests were carried out to find out the effectiveness of solar collector i.e. how many bacteria was removed by solar energy. Also, temperature of sample water were also taken for every two hour during that time period. Similar experimental data were taken for winter and rainy seasons too. The experimental setup for these cases is shown in figure 7.

2.4 Solar Disinfection for Drinking Purpose

2.4.1 Sample Collection and Lab Investigation

From three different ponds, sample water were being collected. The quality parameters of these sample water like pH, turbidity (NTU), color (Pt-Co unit), Fe (mg/l), alkalinity (mg/l as $CaCO_3$), hardness (mg/l as $CaCO_3$), Cl^- (mg/l), CO_2 (mg/l), E. coliform (N/100ml) and Total coliform (N/100ml) were investigated at the lab.

2.4.2 Solar Disinfection using Box Type Solar Collector

As shown in figure 7, experimental setup was done. At first sample water was put in the sedimentation tank for one hour. As a result suspended solids settled on bottom. After that sample water was passed to the box type solar collector. Water was kept in the collector for about 8 hours from 8AM to 4PM. The quality parameters of water was investigated maintaining a time interval of two hours. As a result, the effect of time and temperature on quality parameters was observed. For each of three samples of pond water, similar process was followed.

3. ILLUSTRATION

3.1 Seasonal Variation of Temperature inside the Solar Collector

The maximum temperatures of water inside the box type solar collector along 30 days in summer, winter and rainy season are shown in figure 8. It is seen that the maximum temperature of water inside the solar collector reaches to around 72°C, 60°C and 50°C during summer, winter and rainy season respectively. It is known that bacteria becomes inactive in between 60°C and 65°C (Feroze and Majibur, 2000). The inactivation of bacteria also depends on the timespan i.e. how long the water is at such temperature. From Appendix A, B and C respectively, it is also known that in dry or summer season this time span is 10 AM to 4 PM., in winter season 12 PM to 4 PM and in rainy season this time varies as it becomes cloudy weather now and then. As the intensity of solar energy was less i.e. the temperature of water inside the collector was low in winter and rainy seasons than in the summer, waste water was needed to keep inside the collector for longer time i.e. 12 hours and 16 hours respectively to disinfect bacteria.

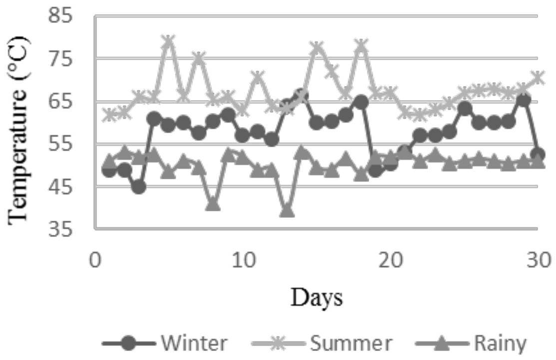

Figure 8: Maximum temperature of sample water

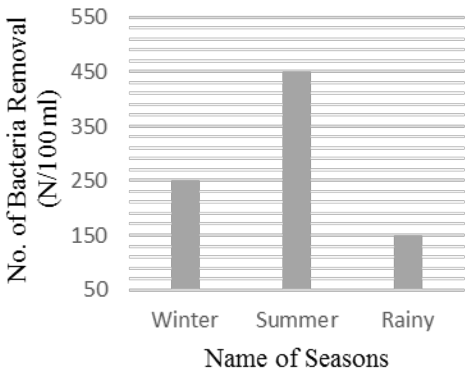

Figure 9: Bacteria removal rate in three seasons

3.2 Bacteria Removal Rate in Different Seasons by the Solar Collector

The bacteria removal rate with respect to 100 ml of sample water from 8 AM to 4 PM i.e. for eight hours in summer, winter and rainy season by the box type solar collector is shown graphically in figure 10. It is seen that no. of bacteria removal is in between 50 to 250 N/ 100 ml, 50 to 450 N/ 100 ml and 150 N/ 100 ml in case of winter, summer and rainy season respectively. It is clear from these data that the solar collector works finest in summer season as it can absorb more and more heat or thermal energy. For the same period of time i.e. eight hours, it is also found that in winter and rainy season no. of bacteria removal is 250 N/ 100 ml and 150 N/ 100 ml whereas in summer this is 450 N/ 100 ml. From table, it is found that in winter, sample water was needed to keep inside the collector for 12 hours to remove 450 N/ 100 ml of bacteria.

Table 1. Minimum Treatment Duration for Micro-organisms in Different Weathers (SODIS, 2010)

Suggested treatment schedule	
Weather conditions	Minimum treatment duration
Sunny (less than 50% cloud cover)	6 hours
Cloudy (50–100% cloudy, little to no rain)	2 days
Continuous rainfall	Unsatisfactory performance, use rainwater harvesting

From table 1, it is found that during 50% -100% cloudy weather with little or no rain, the solar collector can disinfect water. But at the time of continuous rainfall the collector can't disinfect water due to the absence of solar energy. From figure 9, it is observed that in eight hours bacteria removal rate is 150 N/ 100 ml in rainy season.

3.3 Effect of Time and Temperature on Water Quality Parameters

In eight hours with the time interval of two hours, keeping water inside the solar collector, water quality parameters like temperature, pH, turbidity, color, Fe, total coliform, E. coliform etc. were determined for three sample of pond water.

3.3.1 Temperature of Water

Figure 10 illustrates the effect of time on temperature of sample water such that with the increase of time, temperature also increases at first and then decreases. It is also clear that during 12 PM to 4 PM the solar collector can gain maximum thermal energy.

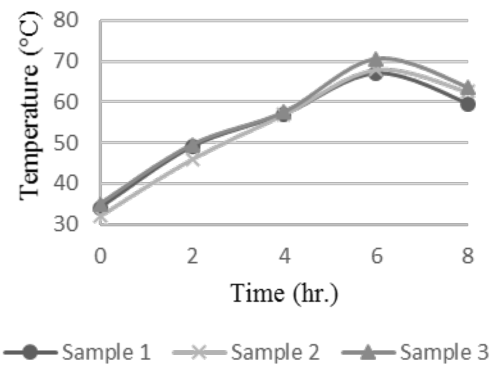

Figure 10: Time (hr.) vs. Temperature (°C) graph

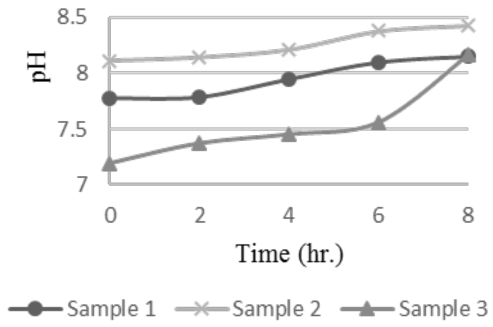

Figure 11: Time (hr.) vs. pH graph

3.3.2 pH Value of Water

Figure 10 and figure 11 reveal that the pH of sample water increases with the increase of temperature. But in each case, the pH of water is within allowable range i.e. 6.5 to 8.5 guide lined by Bangladesh standard (ECR, 1997). So it can be decided the disinfected water is at allowable pH which makes it safe for drinking.

3.3.3 Turbidity of Water

The effect of time on Turbidity of water is shown in figure 12. Raw water used for solar disinfection should be as clear as possible. The allowable range of turbidity is 10 NTU. Bigger particles and solids can be eliminated by letting the particles settle to the bottom in sedimentation tank and afterwards the water is decanted in the collector. From time vs. turbidity graph it is observed that at first the turbidity of water were 3.16, 9.99 and 9.89 NTU which were reduced to 2.17, 5.52 and 5.99 with time for sample one, two and three respectively by letting the particles settle down. Both the initial value and final value of turbidity for all the samples are within allowable limit. So ultimately these sample water is safe to drink in respect of turbidity.

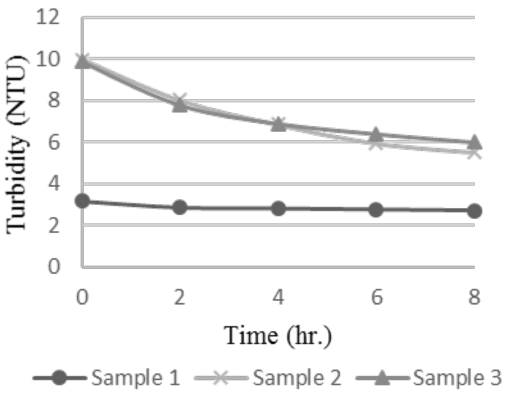

Figure 12: Time (hr.) vs. Turbidity (NTU) graph

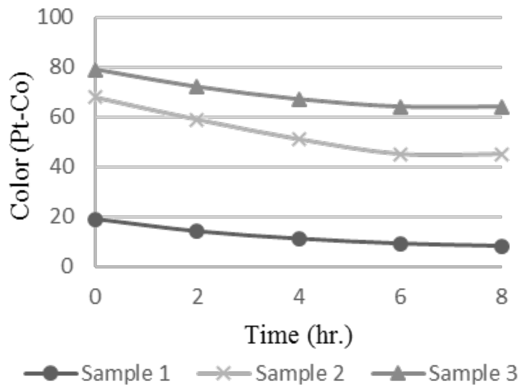

Figure 13: Time (hr.) vs. Color (Pt-Co) graph

3.3.4 *Color of Water*

The relation obtained from the experiment in between time and color is graphically represented in figure 13. The color of sample waters were out of the allowable limit. At initial state the color were 19, 68 and 79 Pt-Co unit of sample one, two and three respectively. After 8 hours these became 8, 45 and 64 Pt-Co unit. As the color of last two sample was still out of allowable range (15 Pt-Co unit), an additional filter was necessary to design to remove color. For this reason a ceramic filter has been added to the outlet pipe of solar collector box and hence it was found that the color was removed successfully.

3.3.5 *Total coliform and E. coliform Bacteria present in Water*

There is an obvious effect of time and temperature on total coliform and E. coliform bacteria present in water. This effect is shown graphically in figure 14 and 15 respectively. Total coliform and E. coliform bacteria are the major impurities in pond water. So it was the prime concern of this study to remove bacteria. The recommended limit for total coliforms and E. coli in drinking water in Bangladesh is zero N/ 100 ml from initial value of total coliform like 137, 120 and 127 N/ 100 ml and E. coliform like 10, 14 and 21 N/ 100 ml in case of three sample.

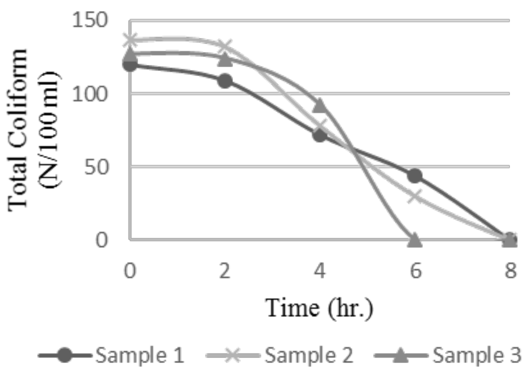

Figure 14: Time (hr.) vs. Total Coliform

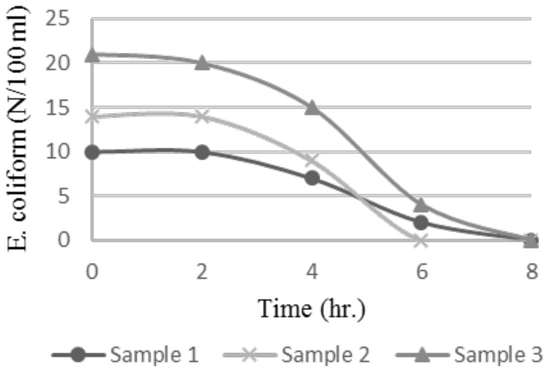

Figure 15: Time (hr.) vs. E. coliform graph

The presence of coliforms is an indication that the water has been contaminated biologically and that it has a risk of containing pathogens. Microorganisms are sensitive to heat. From Time vs. Total coliform and E. coliform graphs show that temperature and exposure time required to eliminate microorganisms. Again it is also found that the solar collector can raise the required temperature with time to remove bacteria successfully. So it can be used for purifying water without any doubt.

From these graphical representation figure 10 to 15, it can be decided that the constructed solar water purifier containing a box type solar collector and a ceramic filter can remove bacteria as well as other impurities. So, this type of water purifier can be used easily with low cost and less maintenance to get pure drinking water.

4. CONCLUSIONS

Now a days it can be said that water is a real luxury. The water crisis leads to tensioned situations and conflicts. Developed countries manage the water crisis through funds for building dams, expensive technologies for recycling water or even desalinizing seawater. Developing countries just like Bangladesh cannot do this. They have to choose between rationalizing water, a fact impeding economic development and the reduction of food production and reuse of the untreated water, a fact that favors the spread of diseases. Providing safe drinking water to the people has become a major challenge for governments in developing countries.

With respect to Bangladesh, it was observed that water purification by solar energy can affect natural resources and economy significantly. Because still now in our country most of the people use boiling phenomena for water purification. For this purpose, they use wood, gas and electricity. Boiling of water requires about 1 kg of wood/liter which also results in deforestation. The fuel like gas and electricity used for this process can be used in other work. On the other hand, solar energy has no limitation and it can be abundantly used in Bangladesh because tropic of cancer is passed over Dhaka, Khulna and Chittagong.

From this thesis work, it is found that the maximum temperature of water inside the box type solar collector could be around 72°C, 60°C and 50°C in dry, winter and rainy season respectively. The solar collector was also capable of removing bacteria in 8 hour during summer, winter and rainy season respectively. With the addition of a ceramic filter, this solar collector box can perform as a solar water purifier appreciably as it maintained other quality parameters of water successfully.

In the dry season water scarcity persists in many areas and in this period surface water is only available in the part of the 22,155 km of major rivers, 1,922 km^2 major standing water bodies and about 1,475 km^2 of pond in the country. At the same time, ground water level is also decreased in significant amount. So in this period solar disinfection of surface water can play a vital role to solve water crisis problem. In the coastal areas in Bangladesh, 70% people use rainwater and 54.8% use water from PSF. For the poor collection and maintenance system, in there significant amount of bacteria is generated. So inactivation of bacteria is necessary in these drinking water sources. The solar collector can play a vital role in this case too.

In a word it can be said that water purification by solar energy is a cheap, simple and potential technology for Bangladesh. This system can be developed day to day and hence can be of great use to us by meeting the demand for pure drinking water.

REFERENCES

Ahmed, M.F. and Rahman, M.M., (2000). “Water supply and sanitation”, Rural and low income urban communication, ITN Bangladesh, center for water supply and waste management, ch-18, page (359-364).

Environmental Conservation Rules (1997). Bangladesh Gazette, Ministry of Environment and Forest, Government of Bangladesh, 27 August 1997.

Feroze, D. A., Mujibur D.R (2000). “Water Supply and Sanitation” Publication ITN- Bangladesh. June, 2000.

Gleick, P.H., ed. (1993). Water in Crisis: A Guide to the World's Freshwater Resources. Oxford University Press. p. 13, Table 2.1 "Water reserves on the earth".

Jagadish, D. A. (1998). "Drinking Water For All" Center for Energy and Sustainable Resources, R.M.K Engineering College Kavaraipettai, India.

Meera, V. and Ahammed, M. M., (2006). water quality of rooftop harvesting systems: a Review. J. water supply Res. Techmol.-Aqua 5 5 (4), 257-268.

SODIS (2010). “How does it work?” Available at http://en.wikipedia.org/wiki/Solar_water_disinfection

UN (United Nation) (2006). "Human Development Report 2006: Beyond Scarcity: Power, Poverty and the Global Water Crisis." United Nations Development Programme, http://hdr.undp.org/en/reports/global/hdr2006/

United Nations Development Programme, (2006). "Human Development Report 2006: Beyond Scarcity: Power, Poverty and the Global Water Crisis." Available at http://hdr.undp.org/en/reports/global/hdr2006/

Walker, D.C., Len, S .V. and Sheehan, B., (2004). Development and evaluation of a reflective pouch for treatment of drinking water. Appl. Environ. Micro biol. 70 (4), 2545 -2550.

WHO/UNICEF Joint Monitoring Programme for Water Supply and Sanitation (2008). "Progress in Drinking-water and Sanitation: special focus on sanitation". MDG Assessment Report 17 July 2008. p. 25.

GREENHOUSE GAS EMISSION SCENARIO OF EXISTING ELECTRICITY GENERATION TECHNOLOGIES IN DHAKA CITY

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ABSTRACT

The future economic development trajectory for Dhaka city is likely to result in a rapid and accelerated growing energy demand, with attendant shortages and problems. .Due to the predominance of fossil resources in the energy mix, there are negative environmental externalities caused by electricity generation. At the same time, the country has very limited fossil fuel reserves to generate electricity. In this context, it is imperative to develop and promote alternative energy resources that can lead to sustainable and environmentally friendly energy systems. There are opportunities for renewable energy technologies for power generation under the climate change regime.They contribute to the mitigation of global climate change through the reduction of GHG emissions, and they confirm to national priorities by leading to the development of local capacities and infrastructure. Energy Ministry of Bangladesh government has introduced a solar energy policy for the housing industry. This policy requires that all newly constructed buildings must include a rooftop solar power unit with an output no less than 3% of the building's total peak load. Mitigation of Greenhouse gas emission through installation of solar panels at Dhaka city residential zone is evaluated using Greenhouse Gas emission guidelines: Stationary combustion sources by North Carolina division of Air quality.Solar installed systems have excellent emission recovery potential which can play a significant role in improvement of climate.The renewable energy resources may serve to supplement the long-term energy needs of Bangladesh to a significant level.

Keywords: Emission, Greenhouse gas, mitigation, renewable energy, solar panel

1. INTRODUCTION

Climate change is one of the most critical global environmental, social, and economic challenges of the century that the entire world is facing.The major challenge for twenty-first century society is and will continue to be the implementation of a global energy production infrastructure that can satisfy total demand, minimize cost, produce as little pollution as possible and extend the lifetime of fossil fuels. New technologies are being developed to reduce the disadvantageous effects of energy production and provide better means to capture and store energy passively. If we were to run out of fossil fuels it could lead to extreme economic strain while an alternative was developed to replace our dependence on these fuels. It could also cause our structured society to collapse in a fight for the remaining fuel. A shortage would cause almost all of our current technologies to become useless.Solar energy has experienced an impressive technological shift. The rapid expansion of the solar energy market can be attributed to a number of supportive policy instruments, the increased volatility of fossil fuel prices and the environmental externalities of fossil fuels, particularly greenhouse gas (GHG) emissions.

Globally, energy sector is identifies as one of the main sector of green house gas (GHG) emissions. Developed industrial nations are responsible for reducing GHG emissions as per Kyoto Protocol. However, the developing nations are not bounded to reduce the GHG emission but there economic development will be influenced by the GHG mitigation strategies of the developed nations. Moreover,the developing nations are expected to get financial and technological support demy Kyoto and post Kyoto phases. Recently, many developing countries have given attention to the study of GHG emissions and finding the alternative paths for GHG mitigation. Like many developing nations, it is important for Bangladesh to estimate the GHG emission level, future trends of GHG emissions and to identify mitigation options. It is important to study and to estimate the GHG emissions of energy sector. It is also needed to identify and to explain the different mitigation options for developing mid term and long-term energy sector of the country. This study presents an estimation of greenhouse gas emission from existing electricity generation technologies and also mitigation of greenhouse gases through solar panel installation at residential area of Dhaka city.

The overall objectives of the paper are:

- Estimating Greenhouse gas emission from conventional electricity generation system provided at the residential area of Dhaka city on fiscal year 2010-2011.
- Estimating mitigation of greenhouse gas after installing solar panel in a residential area of Dhaka city in fiscal year 2010-11.
- Analyzing mitigation of Greenhouse gas emission from conventional power system by installing standard solar panel at Dhaka city from Fiscal year 2008-2009 throughout the life span of solar panel.

2. METHODOLOGY

2.1 CO_2 Emission

The amount of Carbon di oxide emission from electricty genertion sector in the fiscal year 2010-11 at the residential area of the Dhaka city is calculated using calculation based method.Calculation based method is a mass balance approach where the carbon content and carbon oxidation factors are applied to the fuel input levels to determine emissions.

Table 1: Emission factors for calculating CO_2 emissions - fuel analysis approach[1]

Table 1. Emission Factors for Calculating CO_2 Emissions – Fuel Analysis Approach

Fuel Type	Heat Content (Based on HHV)	Carbon Content	Fraction Oxidized
Fossil Fuel Combustion			
Coal and Coke	**MMBtu/ton**	**kg C/MMBtu**	
Anthracite Coal	25.09	28.26	1.00
Bituminous Coal	24.93	25.49	1.00
Sub-bituminous Coal	17.25	26.48	1.00
Lignite	14.21	26.30	1.00
Unspecified (residential/commercial)	22.05	26.00	1.00
Unspecified (industrial coking)	26.27	25.56	1.00
Unspecified (other industrial)	22.05	25.63	1.00
Unspecified (electric utility)	19.05	25.76	1.00
Coke	24.80	31.00	1.00
Natural Gas (by Higher Heating Value)	**MMBtu/scf**	**kg C/MMBtu**	
975 - 1,000 Btu/scf	975 - 1,000 x 10^{-6}	14.36	1.00
1,000 - 1,025 Btu/scf	1,000 - 1,025 x 10^{-6}	14.43	1.00
1,025 - 1,050 Btu/scf	1,025 - 1,050 x 10^{-6}	14.47	1.00
1,050 - 1,075 Btu/scf	1,050 – 1,075 x 10^{-6}	14.58	1.00
1,075 - 1,100 Btu/scf	1,075 – 1,100 x 10^{-6}	14.65	1.00
> 1,100 Btu/scf	> 1,100 x 10^{-6}	14.92	1.00
U.S. Weighted Average (1,029 Btu/scf)	1,029 x 10^{-6}	14.47	1.00
Petroleum Products	**MMBtu/Barrel**	**kg C/MMBtu**	
Asphalt and Road Oil	6.636	20.62	1.00
Aviation Gasoline	5.048	18.87	1.00
Distillate Fuel Oil (#1, 2, and 4)	5.825	19.95	1.00
Jet Fuel	5.670	19.33	1.00
Kerosene	5.670	19.72	1.00
LPG (average for fuel use)	3.849	17.23	1.00
Propane	3.824	17.20	1.00
Ethane	2.916	16.25	1.00
Isobutene	4.162	17.75	1.00
n-Butane	4.328	17.72	1.00
Lubricants	6.065	20.24	1.00
Motor Gasoline	5.218	19.33	1.00
Residual Fuel Oil (#5 and 6)	6.287	21.49	1.00
Crude Oil	5.800	20.33	1.00
Naphtha (<401 °F)	5.248	18.14	1.00
Natural Gasoline	4.620	18.24	1.00
Other Oil (>401 °F)	5.825	19.95	1.00
Pentanes Plus	4.620	18.24	1.00
Petrochemical Feedstocks	5.428	19.37	1.00
Petroleum Coke	6.024	27.85	1.00
Still Gas	6.000	17.51	1.00
Special Naphtha	5.248	19.86	1.00
Unfinished Oils	5.825	20.33	1.00
Waxes	5.537	19.81	1.00
Waste Tires	**MMBtu/ton**	**kg C/MMBtu**	
Waste Tires	28.00	30.77	1.00

Calculation based method is also consists of two approach –
1. Fuel Analysis approach.
2. Generalized approach

In this paper Calculation method-Fuel analysis approach is followed to estimate greenhouse gas emission from the conventional power generation system at the residential area of Dhaka city. The emission is calculated using following equation.

Yearly carbon di oxide emission potential:

$$Emissions_{CO_2} = \sum_{i=1}^{n} Fuel_i \times Heat\ Content_i \times Carbon\ Content_i \times Oxidation\ Factor_i \times \frac{MW_{CO_2}}{MW_C} \times C \ [1]$$

Where,
$\mathbf{Emissions_{CO_2}}$ = Total CO_2 emitted from all fuel types (ton/year)
i = fuel type – bituminous coal , fuel oil , natural gas , etc
$Fuel_i$ = amount of fuel i combusted (ton/year , scf/year or barrels /year)
Heat $Content_i$ = heat content of fuel type i(MMBtu/ton, MMBtu/scf,MMBtu/barrel)
Carbon $Content_i$ = carbon content coefficient of fuel type i (Kg C/MMBtu)
Oxidation $Factor_i$ = farction of fuel type i oxidized (table 1)
MW_{CO_2} = molecular weight of carbon di oxide (44)
MW_C = molecular weight of carbon (12)
C = conversion factor from kg to ton (1 / 907.2)

An emission factor which was used in the emission equation is collected from table 1.

The measurement of total CO_2 emission from the fuel combustion in electricity generation plant in a particular area for a particular fiscal year is done by adding all estimated CO_2 emission from three sources (natural gas, oil and bituminous coal). The whole procedure of estimating emissions is divided in following steps.

2.1.1 Determination of the amount of fuel combusted:

The amount of fuel combusted in electricty generation for dhaka city residential area is derived from the power consumption data of that area in a particular fiscal year 2010-2011 through an indirect method.Electricity generation data is collected from DPDC (Dhaka Power and Distribution Company) which covers south Dhaka city corporation zone and DESCO (Dhaka Electricity Supply Corporation) which covers north Dhaka.
Usually three types of natural resources are used to produce electricity in conventional power system of Bangladesh.

- Electricity production from natural gas sources (% of total) = 89
- Electricity production from coal sources (% of total)= 1.8
- Electricity production from oil sources (% of total)=5

2.1.2 Conversion of the amount of fuel combusted into energy units:

To convert fuel combusted on a mass/volume basis to energy units, the heating value of each fuel type is used. The fuel supplier often provides the heating value of purchased fuel because it is directly related to the useful output of the fuel. Heating value can also be determined through direct fuel sampling and analysis. If heating value data are available, either from the fuel supplier or direct sampling, then that data should be used to represent Heat Content in Equation. Since such data are not available, so default fuel specific heating values listed in Table 3.1 was used.

2.1.3 Determination of carbon content of fuel consumed :

Carbon content of fuel, expressed in terms of mass of carbon per mass, volume or energy of fuel, can also be determined by fuel sampling and analysis. If carbon content data are available, either from the fuel supplier or direct sampling, then that data should be used to represent Carbon Content in Equation. As such data are not available, so U.S. average carbon content coefficients listed in Table 3.1 are used.

2.1.4 *Calculation of Carbon emitted:*

When fuel is burned, most of the carbon is eventually oxidized to CO_2. To account for the small fraction of carbon that may not be oxidized during combustion, the carbon content of fuel is multiplied by the Oxidation factor. As I do not have oxidation factors specific to electricity combustion Sources, default oxidation factor of 1.00 or 100% oxidation was used.

2.1.5 *Conversion of CO_2 emitted to tons:*

To obtain total annual CO_2 emitted in units of tons/year, carbon emissions were multiplied by the ratio of molecular weight of CO_2 to carbon (44/12) and the conversion factor from kg to short tons (1/907.19)

2.2 CH_4 and N_2O Emissions Reporting

Unlike CO_2, CH_4 and N_2O emissions depend not only on fuel characteristics, but also on other factors Such as:

- Technology type, size, age, and efficiency,
- Combustion characteristics
- Use of air pollution control equipment, and
- Routine operational and maintenance practices.

The emissions of methane and nitrous oxide are calculated using following equation, NCDAQ (2009).
Yearly Methane (CH_4) and Nitrous oxide (N_2O) emission potential:

$$Emissions_{CH_4\ or\ N_2O} = \sum_{i=1}^{n} Fuel_i \times Emission\ Factor_i \times C1$$

Where,
$Emissions_{CH_4\ or\ N_2O}$= Total CH_4 and N2O emitted from all fuel type, including bio fuels (ton/year)
i = fuel type - bituminous coal, fuel oil, natural gas, etc.
$Fuel_i$ = amount of fuel type i combusted on energy basis (MMBtu/year)
Emission $Factor_i$ = CH_4 or N_2O emission factors based on fuel type, technology (g/MMBtu) (table 2)
C1 = conversion factor from g to ton (1/907,200)

An emission factor which was used in the emission equation is collected from table 2.

Table 2: Emission factor for calculating CH_4 & N_2O Emission – Generalized Approach [1]

Table 4. Emission Factors for Calculating CH_4 and N_2O Emissions - Generalized Approach

Fuel Type	Sector	CH_4 (g/MMBtu)	N_2O (g/MMBtu)
Coal	Residential	316	1.6
	Commercial	11	1.6
	Industrial	11	1.6
	Electric Power	1	1.6
Petroleum Products	Residential	11	0.6
	Commercial	11	0.6
	Industrial	3	0.6
	Electric Power	3	0.6
Natural Gas	Residential	5	0.1
	Commercial	5	0.1
	Industrial	1	0.1
	Electric Power	1	0.1
Wood	Residential	316	4.2
	Commercial	316	4.2
	Industrial	32	4.2
	Electric Power	32	4.2
Pulping Liquors	Industrial	2.5	2.0

CH_4 and N_2O account for significantly less emissions from stationary combustion than CO_2.The estimation is conducted for three different sources (natural gas, oil and coal) and total amount of CH_4 or N_2O is derived from summing emissions from three different sources.

2.3 Future Greenhouse Gas Emission Scenario at Residential Zone of Dhaka City

At present Bangladesh government has already formulated the Renewable Energy Policy. The government has decided to make mandatory installation of solar panels in case of high-rise buildings having more than 10 floors (consumption level >2 kW) in Dhaka and other major cities in a bid to ease the nagging power crisis. If the high-rise buildings could generate electricity of their own through the solar panels, the circular said, it would reduce pressure on the national grid.At present in both south and north Dhaka certain number of solar panel has been installed by DPDC and DESCO. In this paper, estimation of mitigation of green house gas emission from the conventional power generation system is done where the system is assumed as facilitated by solar panel facility. The estimation is conducted for the life span of a standard solar panel .The data regarding high rise building approval for three recent fiscal year 2009-2010, 2010-2011, 2011-2012 is collected from RAJUK database and information section.This data has been expanded for 25 individual year, which is the standard life span of solar panel ,2009 to 2036 by polynomial extrapolation method. The calcultion is done theoretically installing standard capacity solar panel in each high rise building and estimating their capacity.From measured saved power by solar panel, estimation of greenhouse gas emission is conducted. This amount of greenhouse gas would be emitted if the structure is not facilitated by solar panel facility. Thus the reduced emission by installment of solar panel can be calculated.

3. ILLUSTRATIONS

3.1 GHG Emission From Conventional Power Generation Technology In FY 2010-11

Power consumption data of Dhaka city's residential area in fiscal year 2010-11 is collected from two organizations in Dhaka city who basically distributes power to the consumer. these two organization are:

- Dhaka Power Distribution Company (DPDC)
- Dhaka Electric Supply Company Limited (DESCO)

Total amount of power consumption of residential building of Dhaka city is 3658.125 MkWh. The calculation of amount of fuel combustion is conducted indirectly from amount of power consumed at that area. The process represents how much fuel actually combusted to produce consumed electricity by the residents of Dhaka city on the particular year of 2010-11.before forwarding towards calculation we have to separate source wise electricity generation.

As this Paper topic is concerned about fuel combustion from conventional power generation system so estimation of emission has conducted for three natural fuel sources- natural gas, bituminous coal and oil.

- Electricity production from natural gas sources (% of total) = 89% =3255.73 MkWh
- Electricity production from coal sources (% of total)= 1.8%=6.58MkWh
- Electricity production from oil sources (% of total)=5 %=182.91 MkWh

The amount of carbon di oxide emission is measured using emission equation.
Estimated CO_2 emitted from natural gas = 1.93×10^6 ton/year.
Estimated CO_2 emitted from residual fuel oil =159.810×10^3 ton/year.
Estimated CO_2 emitted from coal = 8787.99 ton/year.

Total amount of carbon di oxide emitted from three major sources (natura gas,bituminous coal, residual furnace fuel oil) to produce electricity which has been distributed to the residential area of Dhaka city in FY 2010-11 is 2.1×10^6 ton/year.
Estimated Methane (CH_4) emission from natural gas =36.42 ton/year
Estimated Methane (CH_4) emission from fuel source oil = 6.084 ton/year
Estimated Methane (CH_4) emission from fuel sourcebituminous coal = 0.094 ton/year

Total amount of methane gas (CH_4) emitted from three major sources (natural gas, bituminous coal, residual furnace fuel oil) to produce electricity which has been distributed to the residential area of Dhaka city in FY 2010-11 is 42.59 ton/year.

Estimated Nitrous Oxide (N_2O) emission from natural gas =3.642 ton/year
Estimated Nitrous Oxide (N_2O) emission from fuel source oil = 1.22 ton/year
Estimated Nitrous Oxide (N_2O) emission from fuel source bituminous coal = 0.15 ton/year

Total amount of nitrous oxide (N_2O) emitted from three major sources (natural gas, bituminous coal, residual furnace fuel oil) to produce electricity which has been distributed to the residential area of Dhaka city in FY 2010-11:
=5.012 ton/year

Final Result:
1.Total amount of carbon di oxide (CO_2) emitted from three major sources is 2.1×10^6 ton/year.
2.Total amount of methane gas (CH_4) emitted from three major sources is 42.59 ton/year
3.Total amount of nitrous oxide (N_2O) emitted from three major is 5.012 ton/year

Table 3: Total GHG emission from conventional electricity generation in FY 2010-11

Emission(ton/year)	Natural Gas	Bituminous coal	Furnace Oil	Total emission (ton/year)
CO_2	1.93×10^6	8787.99	159.810×10^3	2.1×10^6
CH_4	36.42	0.094	6.084	42.59
N_2O	3.642	0.15	1.22	5.012

3.2 Estimating Mitigation of Greenhouse Gas After Installing Solar Panel in a Residential Area of Dhaka City in Fiscal Year 2010-11

The Government of Bangladesh is seeking to expand alternative energy sources as the domestic production of natural gas is on the decline. The Energy Ministry has adopted a multitude of programs to mitigate the reliance on natural gas. Expanding the use of renewable energy has been a major focus area. As a part of this initiative the Energy Ministry has introduced a solar energy policy for the housing industry. This policy requires that all newly constructed buildings must include a rooftop solar power unit with an output no less than 3% of the building's total peak load [2]. In fiscal year 2010-11 both two leading organization Dhaka Power Distribution Company (DPDC) and Dhaka Electric Supply Company limited (DESCO) installed several number of solar panels at the residential area of Dhaka city to lessen the load on conventional electricity supply system as well as to meet the increasing demand of the electricity.After installing solar panel at various residential housing of Dhaka city certain amount of electricity production from conventional power system has been reduced as well as greenhouse gas emission. Electricity provided by solar panels is substituting that certain amount of electricity that is supposed to be supplied by national power grid.The total amount of electricity derived from solar panel facility can be assumed as substitution of conventional power system using natural gas as a fuel. Average capacity of installed single solar panel at south Dhaka by DPDC 1.27 kW and at north Dhaka by DESCO is 0.87 kW in FY2010-11. So, assumed average capacity of hypothetically installed solar panel for reducing future emissionis 1.07 kW. For Dhaka city, sun hour is 7.55 hour and activity time is 365 day [3]

The effect of installment of solar facility at rooftop of Dhaka city's residential area on climate:
Mitigation of carbon di oxide $\mathbf{CO_2}$ = 1462.54 ton/year
Mitigation of methane $\mathbf{CH_4}$ =0.0276 ton/year
Mitigation of nitrous oxide $\mathbf{N_2O}$ = 0.00276 ton/year.
This amount of mitigation is very negligible as it can reduce only small percentage of total emission. If we want to mitigate green house gas emission by installing solar panel we really have to increase the number of solar panel and their capacity.

3.3 Future GHG Emission Reduction Scenario After The Installment Of Solar Panels

Usually the standard life cycle of solar panel is 25 years. So for 25 years it will be capable for mitigating greenhouse gas emission from conventional power generation plant indirectly. Every year certain amount of high-rise building design has been approved by RAJUK. As per government policy it is now mandatory to install solar panel at the roof top of newly constructed high-rise building to .So data has been collected about recent three years building approval from RAJUK. These data is expanded for 25 years from 2009 to 2036 by

polynomial extrapolation method. The total analysis is continued by hypothetically installing standard capacity solar panel at each high-rise building and observing its impact on mitigation of greenhouse gas.Solar panel reduces GHG emission. After installing solar panel in FY 2010-11 the GHG emission will be reduced for 25 years. This recovered future emission is shown in table 4.19. For the year 2013 sample calculation is shown:
Total approval of new building on year 2013 is 3812.
Capacity of newly installed solar panel only approved in that year is 4078.84 kW.
Total capacity of installed solar panel from the year 2009 to 2012 is 12409.86 kW
Thus, present capacity of all solar panel from the year 2009 to 2013 is 16488.7 kW
Total amount of electricity produced by this solar panel: 16488.7 kW×7.55 hour×365 days
= 45438735.03 kWh
Electricity produced using natural gas as a fuel source: 45438735.03 kWh
Amount of natural gas that was used to produce electricity:
= 45438735.03 kWh ×1000 ×0.01003 ft3
= 4.55×108 cubic feet.
= 4.55×108 scf/year
Using equation 1 and table 4.17, the yearly carbon di oxide emission potential was estimated.

Total amount of CO_2 emitted =26899.39 ton/year
Following same procedure total reduction of methane and nitrous oxide gas can also be estimated for study period of 25 year at Dhaka city. This recovered future emission is shown in table 4.

Table 4: Estimation of CO_2 reduction throughout lifespan of standard solar panel

Year	Total approval of Building	Capacity of installed solar panel (kW)	Cumulative Capacity of installed solar panel (kW)	Total kWh/year	Reduced CO_2 emission (ton/year)
2009	2333	2496.31	2496.31	6879206.283	4072.44
2010	2755	2947.85	5444.16	15002743.92	8881.51
2011	3063	3277.41	8721.57	24034466.53	14228.22
2012	3447	3688.29	12409.86	34198471.7	20245.24
2013	3812	4078.84	16488.7	45438735.03	26899.39
2014	4177	4469.39	20958.09	57755256.52	34190.68
2015	4542	4859.94	25818.03	71148036.17	42119.10
2016	4907	5250.49	31068.52	85617073.99	50684.66
2017	5272	5641.04	36709.56	101162370	59887.36
2018	5637	6031.59	42741.15	117783924.1	69727.19
2019	6002	6422.14	49163.29	135481736.4	80204.16
2020	6367	6812.69	55975.98	154255806.9	91318.27
2021	6732	7203.24	63179.22	174106135.5	103069.52
2022	7097	7593.79	70773.01	195032722.3	115457.90
2023	7462	7984.34	78757.35	217035567.3	128483.42
2024	7827	8374.89	87132.24	240114670.4	142146.07
2025	8192	8765.44	95897.68	264270031.7	156445.86
2026	8557	9155.99	105053.67	289501651.1	171382.79
2027	8922	9546.54	114600.21	315809528.7	186956.86
2028	9287	9937.09	124537.3	343193664.5	203168.06
2029	9652	10327.64	134864.94	371654058.4	220016.40
2030	10017	10718.19	145583.13	401190710.5	237501.87
2031	10382	11108.74	156691.87	431803620.8	255624.48
2032	10747	11499.29	168191.16	463492789.2	274384.23
2033	11112	11889.84	180081	496258215.8	293781.12
2034	11477	12280.39	192361.39	530099900.5	313815.14
2035	11842	12670.94	205032.33	565017843.4	334486.30
2036	12207	13061.49	218093.82	601012044.5	355794.59

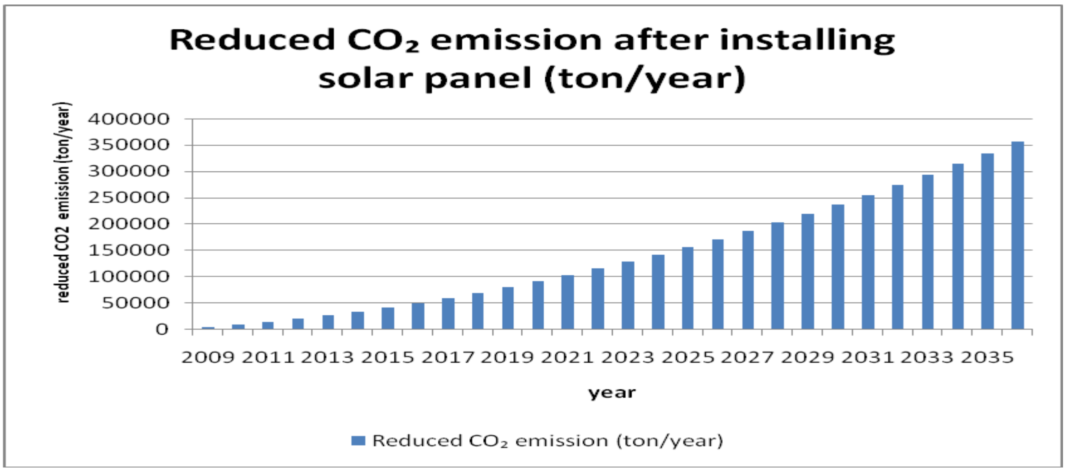

Figure 1 : Mitigation of CO_2 emission throughout lifespan of solar panel

It is evident from figure 1 that the amount of recovered carbon di oxide emission will be increased with the increment of building approval at Dhaka city.

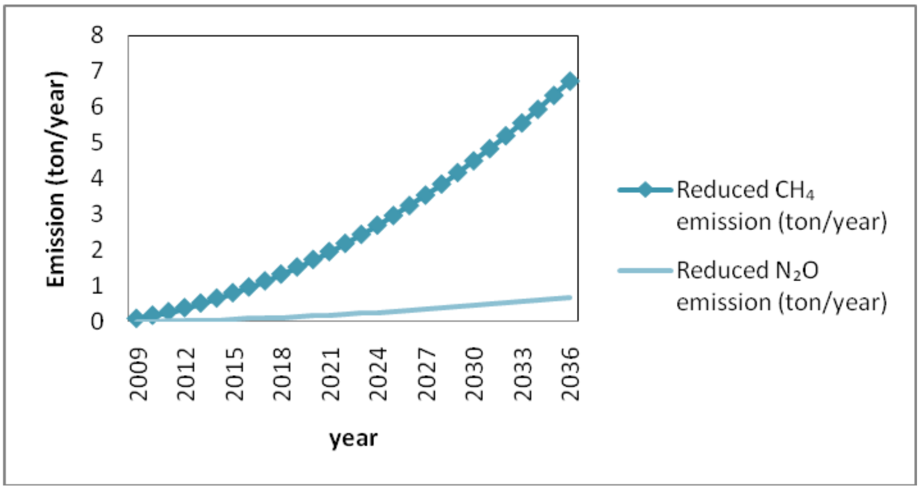

Figure 2 : Mitigation of emission CH_4 and N_2O throughout lifespan of solar panel

In figure 2, reduction of emission CH_4 and N_2O throughout lifespan of solar panel is shown graphically. It is seen that the proportion of nitrous oxide emission is very much low then methane emission. If the capacity and number of solar panel at the roof top of Dhaka city are not increased, presently installed solar panel could not be effective on reducing greenhouse gas emission in long run or throughout its life cycle. So big investment on solar panel installment won't be fruitful.

4. CONCLUSIONS

In Bangladesh, from the conventional electricity generation technologies huge amount of greenhouse gas emits every year. Toxins and particulate matter are released during fossil fuel combustion. This emission has a harmful effect on climate change. The major challenge for twenty-first century society is and will continue to be the implementation of a global energy production infrastructure that can satisfy total demand, minimize cost, produce as little pollution as possible and extend the lifetime of fossil fuels. Solar energy generated via solar panels (also known as Photovoltaic Solar or PV solar) is one of the most sustainable ways we have of generating energy and electricity today. This is one of the most sustainable energy systems which can lessen the increasing demand of electricity at Dhaka city. First and foremost solar panels produce electricity without emissions of any kind. Under the government plan to generate 5 percent of the power from renewable energy sources by 2014, new buildings must install solar panels that could produce 3 percent of the total electricity need of a building before getting any new power connection. To assess the decision taken by government this study presents a detailed analysis on recovered emission scenario by solar systems. It was assumed that each of the buildings will be equipped with a standard capacity solar panel. After installing solar panel at various residential housing of Dhaka city certain amount of electricity production from conventional power system was reduced as well as GHG emission, because solar panel is substituting that certain amount of electricity that was supposed to be supplied by national power grid. From measured saved power by solar panel, greenhouse gas emission was estimated. This is the amount of greenhouse gas that would be emitted if the structure is not equipped with solar panel.

4.1 Findings

A significant amount of GHG gas emitted and will emit from conventional electricity generation system.
It's been shown in this study solar panel production releases much less CO_2 and other greenhouse gases as do electricity generated from coal, oil, natural gas, etc. Solar panel reduces emission from conventional electricity generation technologies indirectly.Moreover, Over Greenhouse gas recovery project by installed solar panel at Dhaka city in FY 2010-11 produce certified emission reductions that can be sold to global carbon market. This emission trading will bring foreign investment that can enhance the solar panel installation practice of Dhaka city. While some changes in the operation of the CDM could increase solar investment, the price of carbon credits required to make solar energy technologies economically competitive with other technologies to reduce GHG emissions would be high. To reduce GHG emission from electricity generation technologies which were conventionally used in Bangladesh, more renewable energy alternatives are required besides solar panels installation. So government should focus on other renewable energy technology."Green Taxes", are currently being levied in various developed nations to balance the difference in cost of renewable and nonrenewable electricity production. These taxes require power companies to pay per ton on emissions they generate. We can estimate the potential of green tax which can be derived from the non-governmental and private power plant which produce electricity for our country. Moreover it can enhance our economy as well as help to reduce greenhouse emission mitigation. Careful monitoring of different components and accurate estimation of emission is very important.The sun exposure is going to be different in other parts of the country, therefore this report and its conclusions are ideal for only Dhaka city.

REFERENCES

[1] NCDAQ (2009), Greenhouse gas emission guidelines: Stationary combustion sources", North Carolina Division of Air Quality, pp 4, http://www.ncair.org
[2] REHAB (2012), "An Evaluation of 3% Roof Top Solar Panel Policy in Bangladesh", Real Estate and Housing Association of Bangladesh, Dhaka, Bangladesh.
[3] Eshita, S.T., Hossain, A. And Raihan, M.O. (2010), "Design and simulation of a solar PV system for BRAC University, Department of Electrical and Electronics, BRAC University, Dhaka, Bangladesh -22.

DOEHLERT DESIGN EXPERIMENT TO OPTIMIZE PH FOR REMOVAL OF TURBIDITY FROM SURFACE WATER BY ELECTROCOAGULATION

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ABSTRACT

Electrocoagulation experiments were carried out in the laboratory in 1.2 litre capacity borosil glass jar over different time intervals with 500 ml of active volume with an operating current of 0.15 ampere with an aluminum electrode. Chemical coagulation has been done by alum solution dosing equivalent amount of aluminum which was dosed during EC process. Turbidity of water was measured by a Nephelometer. Prediction of turbidity removal was modeled by Doehlert Design Experiment. The analysis of variance (ANOVA) and the Regression model was done by MATLAB 7.0 software. In this study surface water has been treated by electrocoagulation to make water potable. Comparative study between ecletrocoagulation (EC) and chemical coagulation (CC) suggested that EC generated aluminum is more active to remove turbidity than CC. EC does not requires pH adjustment during turbidity removal, whereas, CC requires. Doehlert Design Experiment for modeling of turbidity removal w.r. to pH and time at a rate of 0.15 ampere current suggested that optimum removal is obtained at a pH of 6.5 and EC operating time of 50s.

Keywords: Electrocoagulation, chemical coagulation, turbidity, Doehlert Design Experiment, modeling.

1. INTRODUCTION

At the turn of the last century, it was estimated that some 1.1 billion people (one-sixth of the world's population) were without an improved water supply (WHO/ Unicef, 2000), while in the foreseeable future the demand for water is only expected to grow as human population and industrialization increases. The right of all people to access suitable water supplies must thus be seen as a global challenge, the solution of which is dependent on the formulation and implementation of sustainable water management strategies. Microbiological requirements as well as inorganic and organic substances of significance to human health are included. Quality criteria for raw water generally alkalinity, pH, turbidity, pathogen follow drinking-water criteria and even strive to attain them, particularly when raw water is abstracted directly to drinking-water treatment works without prior storage. Drinking water criteria define a quality of water that can be safely consumed by humans throughout their lifetime. Such criteria have been developed by international organizations and include the WHO guidelines for drinking water quality (WHO, 2011).

Coagulation and flocculation are the processes where compounds such as metal salts are added to effluents in order to destabilize colloidal materials. As a result, aggregation of small particles into larger, more easily removed floc takes place. The process of coagulation is largely divided into surface charge neutralization of particles and flocculation by bridging the particles. The effectiveness of the process is influenced by the coagulating agent, the coagulant dosage, the solution pH and ionic strength as well as the concentration and the nature of the organic compounds (Randtke, 1988). Colloid particles do not settle out on standing and cannot be removed by conventional physical treatment processes. Ferric chloride produced better results than alum. Higher dosages did not significantly increase pollutant removal and were not economical. The results provide useful information for raw water treatment. (Koohestanian,et al.2008)

The hydrophobic colloids (clay etc.) pose no affinity for the liquid medium and lack stability in the presence of electrolytes. They are readily susceptible to coagulation. Hydrophilic colloids, such as proteins, exhibit a marked affinity for water. The absorbed water retards flocculation and frequently requires special treatment to achieve effective coagulation (Eckenfelder, 1989).

Electrocoagulation is an electrochemical method of treating polluted water whereby sacrificial anodes corrode to release active coagulant precursors (usually aluminum or iron cataions) into solution. Accompanying electrolytic reactions evolve gas (usually as hydrogen bubbles) at the cathode (Holt et al, 2004). In EC Faraday's law relates the theoretical amount of coagulant generates in the reactor to the current flow (I) in time (t) and is given by the following relation.

$m = ItM/(Z*F)$ (1)

Where,

I = is the current flow, ampere

t = current processing time, second

M = the molecular weight of electrode metal, gm/Mole

Z = the number of electrons transferred

F = Faraday's constant (96,486 C/Mole).

Depending on the solution pH different forms of aluminum species are formed. The basic reactions that take place at the electrodes are presented in the following equations (Vogel, 1979).

$2H^{+} + e = H_2$ (Acid media at cathode) (2)

$H_2O = O_2 + 4H_2 + 4e$ (acid media, at anode) (3)

$H_2O + 2e = H_2 + 2OH^-$ (Basic media, at cathode) (4)

$4OH^- = O_2 + 2H_2O + 4e$ (Basic media, at anode) (5)

Although the electro coagulation mechanism resembles chemical coagulation in that the cationic species are responsible for the neutralization of surface charges, the characteristics of the electro coagulated flock differ dramatically from those generated by chemical coagulation. An electro coagulated flock tends to contain less bound water, is more shear resistant and is more readily filterable. In the EC process the water-contaminant mixture separates into a floating layer, a mineral-rich sediment, and clear water. The aggregated mass settles down due to gravitational force, and is subsequently removed through a drainage valve at the bottom of the EC reaction tank, and moved to a sludge collection tank. The clear, treated water is pumped to a buffer tank for later disposal and/or reuse in the plant's designated process (Keeley, 2012). For low strength synthetic surface water the optimal operational variables were determined as an applied current of I = 2 ampere and a treatment time of 30 minutes. The overall turbidity removal efficiency was found to be 80 % under such conditions (Gunukula, 2008).

In spite of wide application of EC for water and wastewater treatment, the application of EC process for drinking water treatment was not popularly accepted. Thus, in this study the following objectives were addressed.

- Comparative study of EC and CC for removal of turbidity.
- Modeling the effect of pH and coagulant dose to predict turbidity removal.

2. METHODOLOGY

2.1 Materials and Methods

Batch experiments were conducted taking raw Ganga water (Table 1) from Botanical Garden Ghat, Shibpur, West Bengal. A 12V rechargible battery (Exide India Limited) was used to derived the electrocoagulation (EC) reactor. Aluminium plates were used as electrodes. The experiments were carried out in the laboratory in 1.2 litre capacity borosil glass jar over different time intervals with 500 ml of active volume. Chemical coagulation has been done by dosing equivalent amount of alum solution which was dosed during EC process and pH of solution was measured by a pH meter (WTW, Germany). Optimum pH was determined by varying the pH of raw Ganga water with concentrated HCl or NaOH. Turbidity of water was measured by a Nephelometer (Testing Instrument mfg. company, Kolkata). The analysis of variance (ANOVA) and the Regression model was done by MATLAB 7.0 software. All the water quality parameter was measured as per Standard Methods (1995).

Table: 1 The characteristics of raw Ganga water

Parameter	Range	Unit
pH	7.285-7.853	-
Turbidity	210-235	(NTU)
Hardness	179-190	(mg/l as $CaCO_3$)
Alkalinity	206-215	(mg/l as $CaCO_3$)
MPN	109900-462000	(Nos/100 ml)

3. RESULTS AND DISCUSSION

3.1 Effect of Aluminum Dosing

The result of comparative study of electrocoagulation and chemical coagulation is presented in Table 2. The same result is presented in figure 1. It is clear from figure 1 that effluent turbidity becomes 5 NTU at an aluminum dose of 2.5 mg/l by EC. Whereas, to bring down the treated water turbidity below 5 NTU (Indian Standard) it requires 3.35 mg/l of aluminum (obtained from alum). The result suggests that EC generated aluminum is more active than aluminum obtained from alum. It was also found that during CC water requires pH adjustment to achieve maximum turbidity removal. But, in case of EC pH need not to be adjusted because anode splits out water to OH- ion which takes care of required alkalinity. It is also clear from the figure 1 that, after an optimum removal of turbidity, further increase in the aluminum dose, the removal decreased for both EC and CC. This may be due to the fact that at higher aluminum dose the surface charges of the coagulated flocks are reversed and flocks disintegrate, which subsequently increases turbidity.

Dose of Aluminum (mg/l)	Effluent Turbidity (NTU)	
	EC	CC
0.00	210	210
0.84	105	110
1.68	25.5	47
2.52	4.5	9
3.36	3.8	4.9
4.20	5.8	7.2
5.04	8.2	10.7
Current = 0.15 amp, Volume = 500ml		

Table 2: Alum dose vs turbidity removal by EC and CC

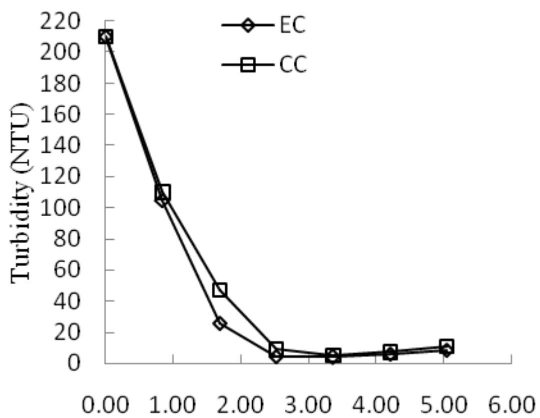

Figure 1: Effect of coagulant on turbidity removal (as aluminum, mg/l)

3.2 Modeling the Effect of pH

Turbidity removal was pH dependent and because EC tends to shift the pH from neutral to slight alkaline pH value so, a separate model had been developed to investigate the effect of pH on turbidity removal. As two level factorial experiments could not model second order response surface, an additional level was required to study for developing second order model. Thus, for two factors there are $3^2 = 9$ runs were required to develop the model. In this section Doehlert Design matrix was used to develop the second order response surface to find optimal pH value. Thus, for two factors instead of 9 experiments only 7 experiments were required. So, it allowed an optimal organization of experiments and, consequently, a reduction of time and cost. Thus to develop the model two parameters namely 'pH' and 'time' was considered. For two variables, the Doehlert Design consists of one central point and six points forming a regular hexagon, and therefore situated on a circle. As it is difficult to maintain pH at some pre-selected value during EC, it was adjusted by adding NaOH or HCl after the EC is over and equilibrated for 30 minutes. The factor level, their natural and coded values and percent turbidity removals are presented in Table 3. It was desired to develop a model that can be represented in the following form.

$$R = \beta_0 + \beta_1 * t + \beta_2 * pH + \beta_3 * t^2 + \beta_4 * pH^2 + \beta_5 * t * pH \qquad (6)$$

where, R is the predicted turbidity removal. β_i values are the regression coefficients are to be determined.
The relationship between coded value (C_v) and the natural value (C_n) is given by equation (7).

$$C_v = C_n - (C_h+C_l)/2]/[(C_h-C_l)/2L_c)] \qquad (7)$$

Where the meanings of the symbols are
C_V= Coded value of the factor,
C_n = Real value or natural value of the factor,
C_h = Upper limit for the natural value,
C_l = Lower limit for the natural value,
L_c = Range of that factor.
Value of L_c is 1 for pH and 0.886 for time.

Table 3: The experimental run for two variables

Run No	Actual value of the factors		Coded value of the factor		Removal (%)
	pH	t(s)	pH	t (s)	
1	6.5	40	0	0	96.38
2	10	40	1	0	50.08
3	8.25	60	0.5	0.866	90.46
4	3	40	-1	0	60.85
5	4.75	20	-0.5	-0.866	75.69
6	8.25	20	0.5	-0.866	70.69
7	4.75	60	-0.5	0.866	85.23

The calculation of coefficients is carried out through the least squares method by MATLAB 7.0 software. The model equation thus obtained can be represented by Eq. (8).

$$R = 96.38 - 3.22 \times pH + 7.89 \times t - 37.92 \times pH^2 - 9.18 \times t^2 + 7.1 \times pH \times t \qquad (8)$$

To obtain optimum (here maximum) value for pH and time (t) Lagrange's criteria was tested. There will be a relative maxima if

$$\Delta = \begin{vmatrix} \frac{\partial^2 R}{\partial pH^2} \cdots\cdots \frac{\partial^2 R}{\partial pH \partial t} \\ \frac{\partial^2 R}{\partial t \partial pH} \cdots\cdots \frac{\partial^2 R}{\partial t^2} \end{vmatrix} \rangle 0$$

The value of Δ is obtained positive (1342.01) and the optimum value is calculated by solving first order derivative equals to zero
i.e. $\partial R/\partial t$ and $\partial R/\partial (pH) = 0$ (9)

Solving equations in (9) we obtained t = 0.4283 and pH = -0.0024 in coded value. In terms of natural value of the parameter the value becomes t = 50 s and pH = 6.5. The value is reasonably in agreement with result reported by literature where turbidity removal is studied by alum coagulation and electrocoagulation by aluminum anode (Bao-yu, 2005). The response surface and contour plot of the model is presented in figure 2.

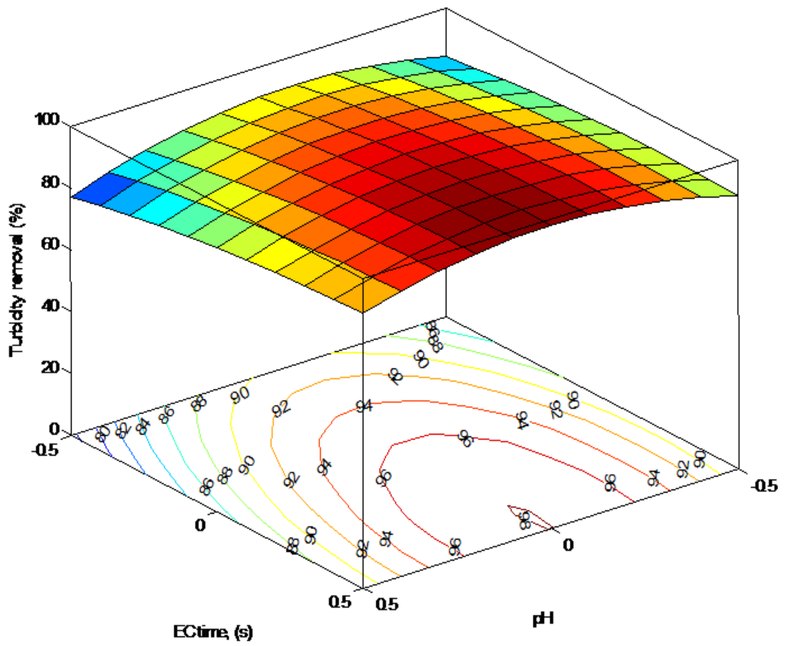

Figure 2: Response surface and contour plot of the model

4. CONCLUSION

In this paper comparative study between electrocoagulation and chemical coagulation has been done. Result suggests that electrocoagulation is more active than chemical coagulation to achieve same turbidity removal. Effect of pH and coagulant dose on turbidity removal has been modeled by Doehlert Design. The optimum pH value obtained was 6.5 with aluminium anode at an operating current of 0.15 ampere and 50s of operating time.

REFERENCES

Bao-yu, G., and Qin-yan, Y. (2005). Natural Organic Matter (NOM) Removal from Surface Water by Coagulation *Journal of Environmental Science Vol.17,No.1, pp.124 – 127,2005*

Eckenfelder. W.W. Jr. (1989). Industrial Water Pollution Control, 3rd Ed.,McGraw Hill, New Delhi.

Gunukula, S. (2011). Electrocoagulation/ Floatation treatment of Synthetic surface water, Thesis for degree of Master of Science in Civil Engineering at the Cleveland and State university.

Holt, P. K., Barton, G.W., and Mitchell, C. A. (2004). The future for electrocoagulation as a localized water treatment technology, Chemosphere 355-367.

Keeley, J.(2012). Electrocoagulation, APHA Recommended Program Guide for Voluntary Sterilization

Koohestanian, A., Hosseini M and Abbasian, Z. (2008). The Separation Method for Removing of Colloidal Particles from Raw Water, American-Eurasian J. Agric. & Environ. Sci., 4 (2): 266-273.

Randtke, S. J. (1988). Organic Contaminant Removal by Coagulation and Related Process Combinations. J.AWWA, 80(5):40-56.

Standard Methods for the examination of water and waste water 1995,APHA, AWWA, WEF.

Vogel, A. I., G. Svehla, J. Bassett. A text book of Quantitative Inorganic Analysis, 6th ed., 1979, Orient Longman, New Delhi.

WHO/Unicef, Source: www.who.int/entity /water_sanitation_health /monitoring /jmp04_3.pdf,2000.

WHO. (2011). Guidelines for drinking-water quality, fourth edition,World Health Organization 2011 Source: http://www.who.int/water_sanitation_health/publications/2011/dwq_guidelines/en/index.html

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A REVIEW STUDY ON USE OF CONSTRUCTED WETLANDS FOR WASTEWATER TREATMENT AND WATER RECYCLING—EXPLORING POSSIBILITY IN BANGLADESH

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ABSTRACT

The rapid population growth in many municipalities in Bangladesh, increasing demands on limited freshwater supplies. The amount of freshwater and the balance among different users significantly affect the development of many cities, including many important region of the country. The population increase has increased not only the freshwater demand but also the volume of wastewater discharged. Next to the development of new management strategies to supply freshwater, the challenge of treating and recycling wastewater will play an important role in the water shortage problem. Use of treated water for irrigating landscapes is often viewed as one of the approaches to maximize the existing water resources and to stretch current urban water supplies. The population forecasts indicate that the population of Bangladesh will be approximately 250 million by 2025. This means there will be a freshwater shortage in the near future if the city does not develop strategies to reclaim or reuse their water. While the environmental and conservational benefits of wastewater reuse are obvious, the major concerns associated with wastewater reuse include effective treatment methods and processes, construction costs, additional costs of installation, and maintenance and management strategies.The need for alternative wastewater treatment systems which are low cost in terms of investment, operation and maintenance especially in the developing countries, is long overdue. The systems required should be the ones that can easily be decentralized and scaled down to small sizes. The systems should also be adapted to the climate and should make use of simple technology and available skills for construction and operation with a possible re-use of the end product. Constructed wetlands (CWs) are in common use in many parts of the world and have been proved to be "cost-effective" methods for wastewater treatment which provide efficient polishing step and an important additional barrier to contaminants in municipal wastewater. Their ability to remove various classes of contaminants, especially wastewater-derived organic compounds are well-documented and can be maximized using appropriate design criteria and experience from ongoing treatment systems. CWs are artificial wetland systems that are designed to exploit the physical, chemical, and biological treatment processes that occur in wetlands and provide for the reduction in organic material, total suspended solids (TSS), nutrients, and pathogenic organisms. In addition to wastewater treatment, wetlands provide additional benefits, including environmental enhancement, habitat for plants and animals and passive recreational opportunities for the community. CWs provide advanced wastewater treatment that is highly valued but of low cost in terms of investment, operation, and maintenance if Compared with the conventional treatment process and are well suited to the environmental conditions in Bangladesh.In this study we have tried to accumulate the integrated approach about the use of Constructed Wetlands in Bangladesh practically and conventionally in a wide range as proven Technology for removal of conventional polutant in varity of wastewater considering geogrphical condition, climatic condition and polutant loading which can be the sustainable solution for wastewater treatment as well as reuse of water resource of global village for any developing country like Bangladesh.

Keywords: Wetlands, Constructed Wetlands (CWs), Biological Treatment, Cost-effective, Water Reuse..

1. INTRODUCTION

Wetlands are defined as land where the water surface is near the ground surface lung enough each year to maintain saturated soil conditions, along with the related vegetation. Marshes, bogs, and swamps are all examples of naturally occurring wetlands. A "constructed wetland" is defined as a wetland specifically constructed for the purpose of pollution control and waste management, at a location other than existing natural wetlands. Constructed wetlands are essentially inspired by natural processes in naturally occurring wetlands. Natural wetlands—marshes, swamps, bogs, everglades—are highly productive ecosystems storing large volumes of water, recirculating nutrients, providing habitats that support a diverse population of plants and animals, and removing pollutants from the water. Wetlands, under natural conditions, are capable of providing significant pollutant removal from influent streams of impaired waters—including stormwater, municipal and industrial wastewater, landfill leachates, and urban runoff. Treatment in wetlands occurs as a result of settling of suspended particles, oxidation of organic matter, metabolic activity by indigenous microorganisms, photolysis, and uptake of nutrients by plants growing within the wetlands. As long as the wetland is not overloaded, dried out, or subjected to extended cold periods, effluent quality can be expected to be suitable for most non-potable uses. There are two basic types of constructed wetlands, the free water surface wetland and the subsurface flow wetland (Figure- 1). Both types utilize emergent aquatic vegetation and are similar in appearance to a marsh. The free water surface (FWS) wetland typically consists of a basin or channels with some type of barrier to prevent seepage, soil to support the roots of the emergent vegetation, and water at a relatively shallow depth flowing through the system. The water surface is exposed to the atmosphere, and the intended flow path through the system is horizontal. The subsurface flow (SF) wetland also consists of a basin or channel with a barrier to prevent seepage, but the bed contains a suitable depth of porous media Rock or gravel are the most commonly used media types in the U.S. The media also support the root structure of the emergent vegetation. The design of these systems assumes that the water level in the bed will remain below the top of the rock or grave media. The flow path through the operational systems in the U.S. is horizontal.

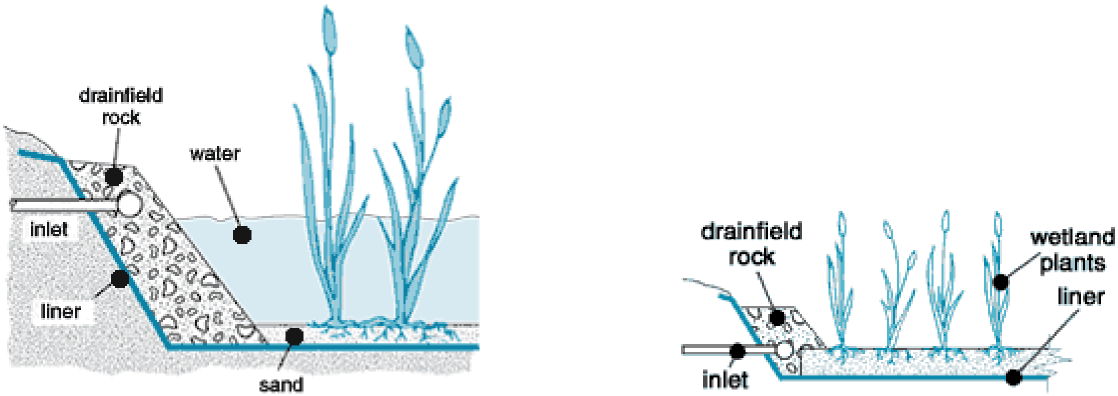

Figure- 1: Two Types of Constructed Wetlands: Free Surface Flow (FSF), on the left and Subsurface Flow (SSF), on the right side of the figure

2. METHODOLOGY

The utilization of constructed wetlands (CW) in water pollution control provides an alternative perspective that is based on the water quality functions and values of natural wetlands but which is not limited by legal and conservation regulations. Constructed Wetlands is a biological wastewater treatment technology designed to mimic processes found in natural wetland ecosystems. These systems use wetland plants, soils and their associated microorganisms to remove contaminants from wastewater. CWs are receiving increasing worldwide attention for wastewater treatment and recycling due to the following major advantages:

- Use of natural processes
- simple construction (can be constructed with local materials)
- simple operation and maintenance
- cost-effectiveness (low construction and operation costs)
- Process stability.

2.1 Types of constructed wetlands

Constructed wetlands can be classified according to their hydrologic and water flow regimes. The three main types of constructed wetlands currently used are described below, in all cases a plastic or clay liner is usually used to prevent wastewater seepage or underground infiltration into the systems. Free Water Surface (FWS) CWs mimic natural marshes having areas of open water and areas of macrophytes. Floating, submerged or emergent vegetation can be present in FWS wetlands. These systems are also called surface flow wetlands. They are similar to aerobic ponds, usually shallow and having the water flowing through the stands of vegetation (Figure2.1). Horizontal subsurface flow (HF) CWs consist of gravel or coarse sediment beds planted with emergent macrophytes. The water is kept below the surface of the gravel and it flows horizontally through the media and plant roots from the inlet to the outlet area. These systems are also known as reed beds or root zone method (Figure 2.2).

Vertical Flow (VF) CWs consist of sand or gravel beds planted with emergent macrophytes. The water is distributed on the surface of the bed and then percolates through the media down to the outlet zone usually placed on the bottom of the bed (Figure 2.3). VF wetlands can also be operated in an up-flow manner, by inverting the position of inlet and outlet pipes, or in a tidal fashion, with fill and drain cycles.

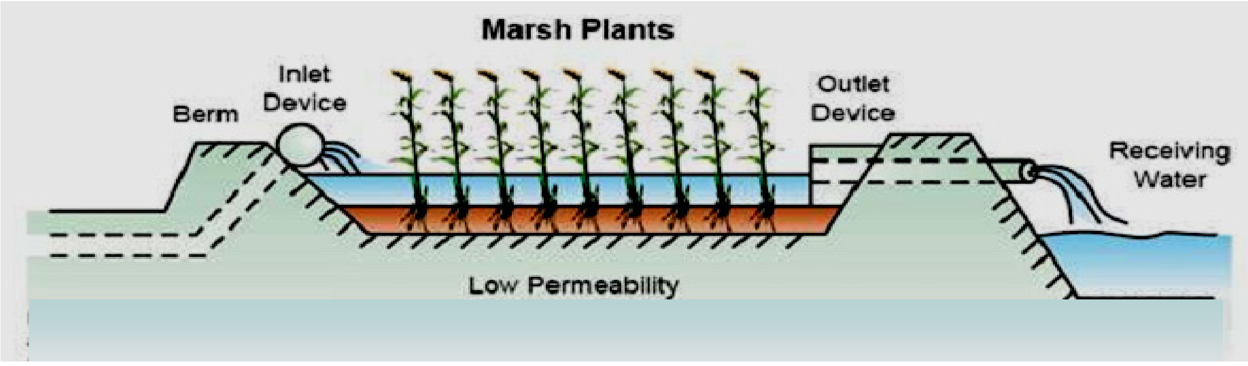

Figure- 2.1 : Section view of a typical FWS wetland (Extracted from Kadlec and Knight, 1996).

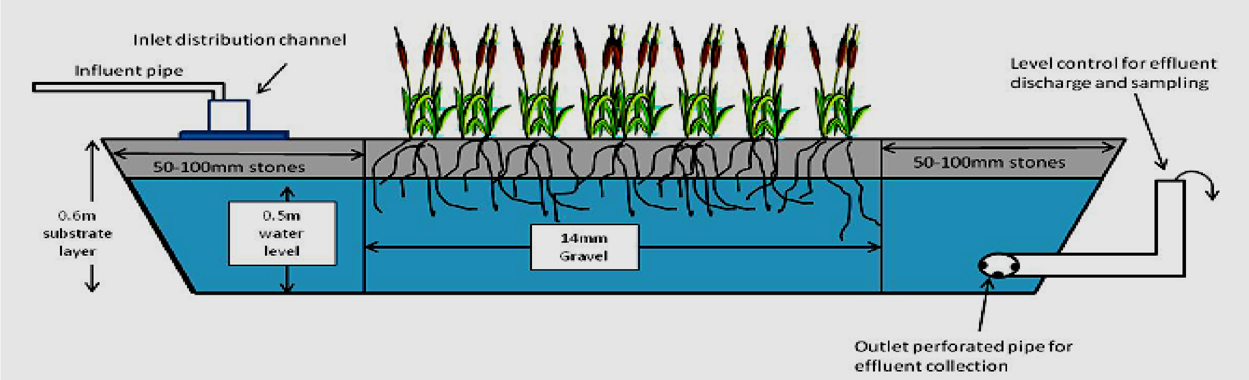

Figure- 2.2: Section view of a typical HF wetland with details of major components.

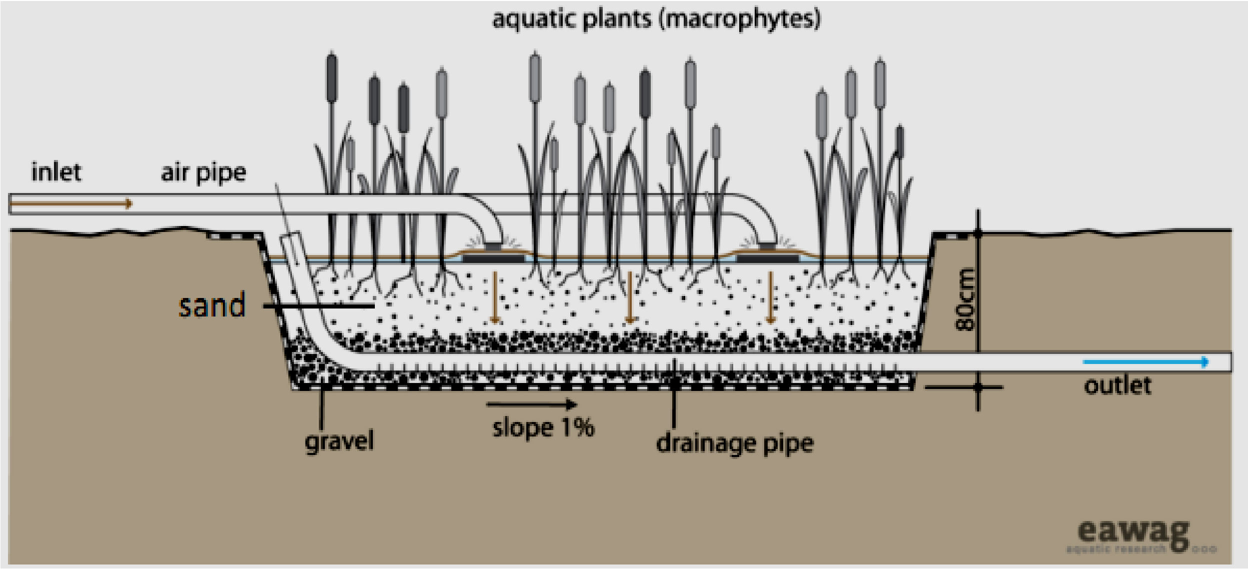

Figure- 2.3: Section view of a typical VF wetland with details of major components

2.2 Constructed wetland wastewater treatment system

In extremely hot and dry climates, a gravel or sand bed can help minimize evaporative loss of water. A mix of aquatic plant materials (cattails, bulrushes, reeds, etc.) growing either hydroponically or in a gravel substratum provide the media through which polluted water is filtered and biologically treated. In the FSF type of wetland, the impoundment is constructed with an impermeable bottom and sides and the inlet allows polluted water into the pond, diffused from several points through a gravel embankment. This type of constructed wetland closely mimics natural wetlands in terms of vegetation and water flow regime. Vegetation covers most of the water surface, but some areas are left bare to maximize photolytic processes and to provide habitat for birds and access to recreational users of the wetland. Wastewater flows over the soil and through the thicket of vegetation rooted in the soil. Pollutant removal is achieved with sedimentation, filtration, oxidation, reduction, adsorption, and precipitation. Pollutant concentration decreases linearly or exponentially along the length of the flow path. Constructed FSF wetlands are adapted well for temperate and warm regions—such as in Bangladesh. In the SSF type of constructed wetland, water flows below ground, or subsurface, through gravel or other media. These systems can be designed for horizontal or vertical flow through the media. In vertical-flow systems, underdrains are installed below the rooting media for collection of the effluent. Microbial populations adapted to the anaerobic environment provide for anoxic decomposition. Because of the larger surface area of exposure in SSF systems, greater densities of bacteria can work on some of the pollutants, such as BOD and organic compounds, resulting in a smaller footprint than the comparable FSF system. However, the effectiveness of the two systems is similar in removal of nitrogen, phosphorus, suspended solids, and trace metals. Flow through the SSF wetlands is slower than in the FSF wetlands because of the friction encountered in flowing through the media (sand, soil, gravel) and root systems of the aquatic plant. The additional removal benefit provided by the roots within the media is rather limited. Impermeable lining of the wetlands is necessary if it is desired to maximize the volume of treated effluent (TSE) for reuse. The SFF wetlands are better suited to cold climates and where Vertical SSF wetlands provide the opportunity for both aerobic and anaerobic treatment by intermittent draining and filling of the media. During the drainage phase, air becomes entrapped within the porous media, creating many small pockets, which will stay aerobic after refilling of the wetland until the next cycle of drainage. Research studies have shown that wetland systems have great potential in controlling water pollution from domestic, industrial and non-point source contaminants. As it has been widely recognised as a simple, effective, reliable and economical technology compared to several other conventional systems, it can be a useful technology for developing countries like Bangladesh. Constructed wetlands remove pollutants from wastewater through various physical, chemical and biological mechanisms. Some of the main pollutant removal mechanisms in constructed wetlands are presented in Table- 2.1.

Table- 2.1: Pollutant removal mechanisms in constructed wetlands.

Wastewater constituents	**Removal mechanism**
Suspended solids	Sedimentation Filtration
Soluble organics	Aerobic microbial degradation Anaerobic microbial degradation
Phosphorous	Matrix sorption Plant uptake
Nitrogen	Ammonification followed by microbial nitrification Denitrification Plant uptake Matrix adsorption Ammonia volatilization (mostly in SF system)
Metals	Adsorption and cation exchange Complexation Precipitation Plant uptake Microbial oxidation/reduction
Pathogens	Sedimentation Filtration Natural die-off Predation UV irradiation (SF system) Excretion of antibiotics from roots of macrophytes

Source: Cooper et al., 1996

2.3 Constructed wetland Design Criteria

Various types of constructed wetlands differ in their main design characteristics as well as in the process which are responsible for pollution removal. The most critical component of constructed wetlands is land. Because of the relatively large surface area necessary for a given flow, use of constructed wetlands is generally limited to smaller communities, usually down to single-family or single-building settings. The land is shaped into basins, commonly (but not always) lined with impermeable membranes and sloped to provide a gentle flow from the inlet to the outlet of the treatment system. A gravel lining is built above the liner and it can vary in thickness from a minimal root zone to varying depths that might extend to above the water line.

2.3.1 Free Water Surface Constructed Wetlands

A typical FWS CW with emergent macrophytes is a shallow sealed basin or sequence of basins, containing 20-30cm of rooting soil, with a layer depth of 20-40cm. Dense emergent vegetation covers a significant fraction of the surface, usually more than 50% (Figure- 2.3.1). Besides planted macrophytes, naturally occurring species may be present. Plants are usually not harvested and the litter provides organic carbon necessary for denitrification which may proceed in anaerobic pockets within the litter layer. Sizing of FWS CWs is usually based either on volume or area. Volume-based methods use a hydraulic retention time to assess the pollutant removal while area-based methods assess pollutant reduction using the overall wetland area. In table 2.3.1, the basic sizing criteria for BOD_5, TSS and TKN removal are given.

Table- 2.3.1: Loading rates recommended for achieving target effluent concentration in FWS CWs. [5]

Parameter	Effluent	Loading rate
BOD_5	30 mg/L 25 mg/L 20 mg/L	6.0 $g/m^2 d$ 3.0 $g/m^2 d$ 4.5 $g/m^2 d$
TSS	30 mg/L 30 mg/L 25 mg/L 20 mg/L	7.0 $g/m^2 d$ 5.0 $g/m^2 d$ 3.5 $g/m^2 d$ 3.0 $g/m^2 d$
TKN	10 mg/L	1.5 $g/m^2 d$

Figure 2.3.1: A FWS CW for tertiary treatment of municipal wastewater in McGrath Hill, Hawkesbury, near Sydney, NSW, Australia.

2.3.2 Horizontal Surface Flow Constructed Wetlands

HF CWs consist of gravel or rock beds sealed by an impermeable layer and planted with wetland vegetation (Figure- 2.3.2). The wastewater is fed at the inlet and flows through the porous medium under the surface of the bed in a more or less horizontal path until it reaches the outlet zone, where it is collected and discharged. In the filtration beds, pollution is removed by microbial degradation and chemical and physical processes in a network of aerobic, anoxic, anaerobic zones with aerobic zones being restricted to the areas adjacent to roots where oxygen leaks to the substrate. Despite problems with surface flow soil-based systems exhibited high treatment effect for organics and suspended solids if reed bed area 3–5 m^2 PE^{-1} (population equivalent) was used [6]. In the late 1980s, soil material was replaced by coarse material and at present, washed gravel or rock with grain size of about 10–20 mm are commonly used [7]. For a long time, the HF CWs have been designed using either simple "rule of thumb" set at 5 m^2 PE−1 or plug-low first order models [8]. Recently, more complex dynamic, compartmental models have been developed. However, in these models many parameters are difficult to measure and therefore many assumptions must be made. Hence, it is important to realize that more complex models do not necessarily bring more precise design parameters. However, no matter which design model is used, for municipal sewage, the area of HF CWs is usually about 5 m^2 PE^{-1} [7]. To achieve the outflow BOD5 and TSS concentration of 30 mg/L, the U.S. EPA recommends the respective inflow loads of 6 g/m^2 d and 20 g/m^2 d.

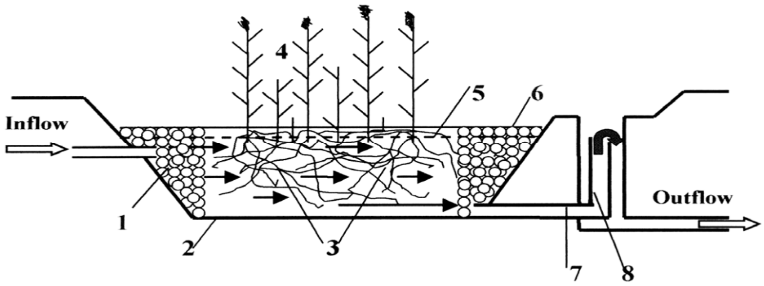

Figure- 2.3.2: Schematic layout of a constructed wetland with horizontal subsurface flow. 1 inflow distribution zone filled with large stones; 2 impermeable layer; 3 filtration material; 4 vegetation; 5 water level in the bed; 6 outflow collection zone; 7 drainage pipe; 8 outflow structure with water level adjustment.

2.3.3 Vertical Surface Flow Constructed Wetlands

Vertical flow constructed wetlands (VF CWs) (Figure- 2.3.3) were originally introduced by Seidel to oxygenate anaerobic septic tank effluents [9]. However, the VF CWs did not spread as quickly as HF CWs probably because of the higher operation and maintenance requirements due to the necessity to pump the wastewater intermittently on the wetland surface. The water is fed in large batches and then the water percolates down through the sand medium. The new batch is fed only after all the water percolates and the bed is free of water. This enables diffusion of oxygen from the air into the bed. VF CWs are also very effective in removing organics and suspended solids. Removal of phosphorus is low unless media with high sorption capacity are used. As compared to HF CWs, vertical flow systems require less land, usually 1–3 m^2 PE^{-1} [10-13]. The early VF CWs were composed of several stages with beds in the first stage fed in rotation. At present, VF CWs are usually built with one bed and the system is called "compact" VF CWs [11]. In upflow vertical CWs, the wastewater is fed on the bottom of the wetland. The water percolates upward and then it is collected either near the surface or on the surface of the wetland bed. Recently, the "fill and drain" or "tidal" CWs have been developed. In tidal flow systems the wastewater percolates upwards until the surface is flooded. When the surface is completely flooded, the feeding is stopped, the wastewater is then held in the bed and, at a set time later, the wastewater is drained downwards. After the water has drained from the filtration bed, the treatment cycle is complete and air can diffuse into the voids in the filtration material [14].

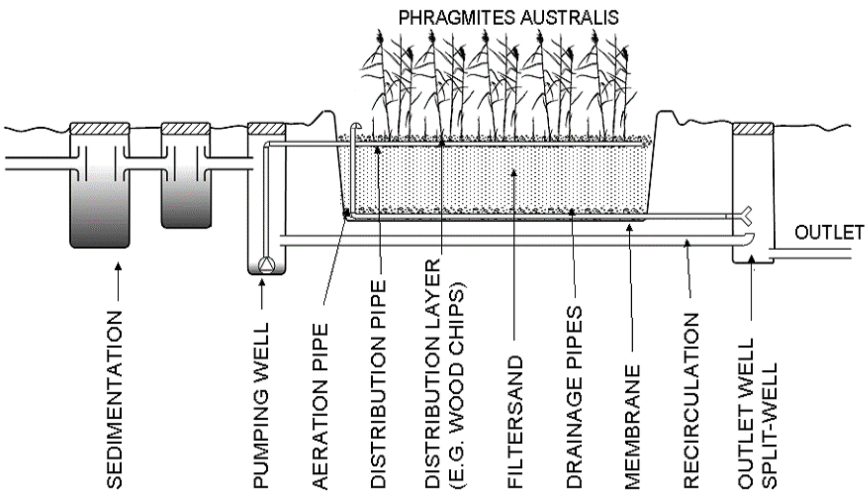

Figure- 2.3.3: Layout of a vertical flow constructed wetland system for a single household. Raw sewage is pre-treated in a sedimentation tank. Settled sewage is pulse-loaded onto the surface of the bed by a level-controlled pump. Treated effluent is collected in a system of drainage pipes, and half of the effluent is re-circulated back to the pumping well (or to the sedimentation tank).

3. RESULTS AND DISCUSSION

3.1 Treatment Efficiency

Removal of organics is high in all types of constructed wetlands as experimented by Vymazal and Kröpfelová (Table- 3.1). While in FWS and VF constructed wetlands, the microbial degradation processes are mostly aerobic, in HF constructed wetlands, anoxic and anaerobic processes prevail. The results presented in Table- 3.1 below also indicate that hydraulic retention time is usually lower in FWS CWs as compared to sub-surface flow CWs.

Table- 3.1: Treatment efficiency (Eff, in %) of various types of constructed wetlands (CWs) for organics and suspended solids. Inflow (In) and outflow (Out) concentrations in mg/L. HLR = hydraulic loading rate (cm/d). N = number of CWs. [7]

Type of CW	BOD_5					TSS				
	In	Out	Eff	HLR	N	In	Out	Eff	HLR	N
FWS	161 34.6	42 9.8	74 72	4.1 3.3	50 51	185 57.8	43 18.3	77 68	4.8 3.1	52 52
HF	170	42	75	11.8	438	141	35	75	15.4	367
VF	274	28	90	8.2	125	163	18	89	9.7	98

Removal of nutrients in various types of constructed wetlands is presented in Table- 3.2 below. Phosphorus retention is low in all types of constructed wetlands and CWs are seldom built with phosphorus being the primary target of the treatment. Most studies on phosphorus cycling in wetlands have shown that soil/peat accumulation is the major long-term phosphorus sink [17]. Among the various types of constructed wetlands, soil accretion occurs only in FWS CWs as the vegetation is not harvested and wastewater gets in contact with top soil layer. In sub-surface flow CWs, the major removal mechanisms are adsorption and precipitation. The removal of phosphorus is very high with these substrates, but it is important to realize that sorption and precipitation are saturable processes and the sorption decreases over time.

Removal of total nitrogen (Table- 3.2) is also usually low due to low nitrification in water-saturated HF constructed wetlands and low or zero denitrification in FWS and VF CWs, respectively [7, 15, 16]. In FWS CWs nitrogen is removed via nitrification in aerobic water column and subsequent denitrification in anoxic litter layer on the bed surface.

In order to achieve effective removal of total nitrogen VF CWs could be combined with HF CWs which, in contrast, do not nitrify but provide suitable conditions for reduction of nitrate formed during nitrification in VF beds. Plant uptake in all types of constructed wetlands is effective only when plants are harvested, but the amount sequestered in the aboveground biomass is usually very low and does not exceed 10% of the inflow nutrient load [7].

Table- 3.2: Treatment efficiency (Eff, in %) of various types of constructed wetlands (CWs) for nitrogen and phosphorus. Inflow (In) and outflow (Out) concentrations in mg/L. HLR = hydraulic loading rate (cm/d). N = number of CWs. [7, 15, 16]

Type of CW	TP					TN					NH_4-N				
	In	Out	Eff	HLR	N	In	Out	Eff	HLR	N	In	Out	Eff	HLR	N
FWS	14.7 4.0 7.9 3.6	9.7 1.8 5.1 1.8	34 49 35 30	5.4 12.3 3.5	52 207 282 52	42.6 11.7 84 10.9	23.5 6.2 49.5 4.6	45 47 41 58	4.9 8.9 3.2	29 192 116 36	30 75 5.8	16 46 2.7	48 39 53	5.4 7.3 3.1	40 118 59
HF	9.6	4.8	50	11.4	272	63 54	36 36	43 33	10.6 7.6	208 123	36 40	22 28	39 30	14.1 7	305 213
VF	10.3	4.5	56	8.2	118	73	41	43	9.1	99	56	14.9	73	8.4	129

3.2 Exploring Possibilities in Bangladesh

Recycled water coming from the wetland/ constructed wetland can be safely used for a variety of purposes in Bangladesh. Which are as follows:

- Residential garden watering.
- Irrigation to agricultural and horticultural crops.
- Closed system toilet flushing.
- Processed (partially treated) / cooling water for industry.
- The fire protection stores and reticulation systems.
- Irrigation of municipal parks and sports ground.
- Water for ornamental ponds.
- Water for cleaning purpose in dry season (like road, garage, parking lot).

Also in small-scale, compact, constructed wetlands can be implemented for use within commercial buildings in Bangladesh especially for reuse of treated wastewater or graywater for landscaping which is being marketed by several manufacturers in different country. One example is the so-called "Living Machine", marketed by Worrell Water Technologies (Worrell, 2011) and represented in cross-sectional view in (Figure-3.1) below.

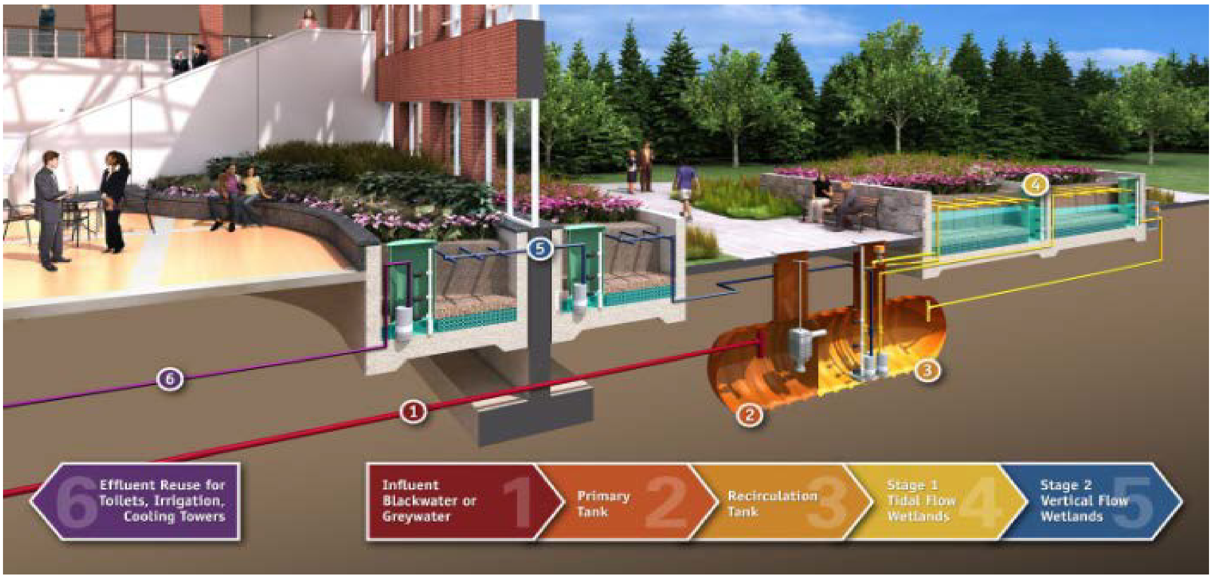

Figure- 3.1: Worrell's Living Machine—Constructed Wetlands in An Upscale Office Building

That's why for effective exploring possibility in Bangladesh, we are trying to implement an experimental study on constructed wetland technology to recycle and reuse of domestic wastewater coming from a student residential hall namely "Dr. F. R. Khan Hall" situated in the campus of Dhaka University of Engineering and Technology, Gazipur- 1700, Bangladesh which can be the preliminary effective step for sustainable solution of wastewater management and control.

4. CONCLUSIONS

It has been concluded from this study, the constructed wetlands are very effective for wastewater treatment in Bangladesh which can provide the near-final step for producing an effluent that can meet advanced water treatment standard. Water produced at the discharge point of constructed wetland is useful for a wide variety on non-potable application in Bangladesh if it could be enhanced by using a combination of various types of constructed wetland. It is noticed that constructed wetlands required very low/ zero energy input and therefore the operation and maintenance cost are much lower compared to conventional treatment system in Bangladesh. Though it has some construction cost primarily but that cost is also lower if compared to construction cost of the conventional treatment system. If it is possible to explore this cost effective technology in wide range in Bangladesh then it will play an important role in indirect potable reuse where it is the part of overall system also which may provide other ecosystem services such as flood control, carbon sequestration or wild life habitat, ice melting issue, sea level rising as well as reducing the impact of climate change on human being.

REFERENCES

1. Seidel, K. Die Flechtbinse Scirpus lacustris. In Őkologie, Morphologie und Entwicklung, ihre Stellung bei den Volkern und ihre wirtschaftliche Bedeutung; Schweizerbart´sche Verlagsbuchnadlung: Stuttgart, Germany, 1955; pp. 37-52.

2. Sheikh B, WateReuse Research Foundation Publication 05-006-1

3. Cooper P F, Job G D, Green M B and Shutes R B E (1996) Reed Beds and Constructed Wetland for Wastewater Treatment, WRc Swindon, UK

4. Kadlec, R.H. Overview: Surface Flow Constructed Wetlands. In Proceedings of the 4th International Conference Wetland Systems for Water Pollution Control; ICWS Secretariat: Guangzhou, China, 1994; pp.1-12.

5. Wallace, S.D.; Knight, R.L. Small Scale Constructed Wetland Treatment Systems. Feasibility, Design Criteria, and O&M Requirements; Water Environ. Res. Foundation: Alexandria, VA, USA, 2006.

6. Brix, H.; Schierup, H.-H. Sewage treatment in constructed wetlands—Danish experience. Wat. Sci. Tech. 1989, 21, 1665-1668.

7. Vymazal, J.; Kröpfelová, L. Wastewater Treatment in Constructed Wetlands with Horizontal Sub-Surface Flow; Springer: Dordrecht, The Netherlands, 2008.

8. Kadlec, R.H.; Wallace, S.D. Treatment Wetlands, 2nd ed.; CRC Press: Boca Raton, FL, USA, 2008.

9. Seidel, K. Neue Wege zur Grundwasseranreicherung in Krefeld, Vol. II. Hydrobotanische Reinigungsmethode. GWF Wasser/Abwasser 1965, 30, 831-833.

10. Brix, H. The use of vertical flow constructed wetlands for on-site treatment of domestic waste water: New Danish guidelines. Ecol. Eng. 2005, 25, 491-500.

11. Cooper, P.F. A review of the design and performance of vertical flow and hybrid reed bed treatment systems. Wat. Sci. Tech. 1999, 40, 1-9.

12. Weedon, C.N. Compact vertical flow reed beds: design rationale and early performance. IWA Macrophytes Newsletter 2001, 23, 12-20.

13. Molle, P.; Liénard, A.; Boutin, C.; Merlin, G.; Iwema, A. How to treat raw sewage with constructed wetlands: an overview of French systems. Wat. Sci. Tech. 2005, 51, 11-21.

14. Cooper, P.F. The performance of vertical flow constructed wetland systems with special reference to the significance of oxygen transfer and hydraulic loading rates. Wat. Sci. Tech. 2005, 51, 81-90.

15. Kadlec, R.H.; Wallace, S.D. Treatment Wetlands, 2nd ed.; CRC Press: Boca Raton, FL, USA, 2008.

16. Vymazal, J. Removal of nutrients in various types of constructed wetlands. Sci. Tot. Environ. 2007, 380, 48-65.

17. Richardson, C.J. Mechanisms controlling phosphorus retention capacity in freshwater wetlands. Science 1985, 228, 1424-1427.

18. Routledge, E. J.; Sheahan, D.; Desbrow, C.; Brighty, G. C.; Waldock, M.; Sumpter, J. P. Identification of estrogenic chemicals in STW effluent. 2. In vivo responses in trout and roach. Environ. Sci. Technol., 1998 32, 1559–1565.

19. WateReuse Research Foundation, Evaluate Wetland Systems for Treated Wastewater Performance to Mee Competing Effluent Water Quality Goals, Publication Number 05-006-1, Washington, D. C., USA, 2011.

20. Worrell, 2011: http://www.livingmachines.com/about/

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E-WASTE RECYCLING PRACTICES IN BANGLADESH

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ABSTRACT

The electrical and electronics industry is one of the world's fastest growing manufacturing sectors. As a result of this rise in production, as well as the increasing rate of product obsolescence, waste from electrical and electronic equipment, or e-waste, has become the fastest-growing waste stream in the (post-) industrialized world. Sustainable management for e-waste recycling is a concerning fact in Bangladesh to lessen environment pollution and health problems, which are attracting growing public interest. This study helps to scrutiny the existing e-waste management process and introduces proposals for the sustainable management of e-waste products including the potentials of best possible recycling in the existing system. A structured questionnaire has been processed in Dhaka and Chittagong and existing literature were reviewed. Around 1,20,000 urban poor from the informal sector have been found to be involved in the e-waste recycling trade chain in Dhaka city area where only children (under 10 year of Age) accounted for approximately 50,000; Amongst them about 40% were involved in ship breaking yards. Workers in the recycle shops were found to be less paid with monthly wages approximately BDT 3000 while a day labourer earns almost double approximately BDT 200 per day. The results of the study would provide us important insight into the growing concern of e-waste and would help us to gather input for designing policy measure to recycle e-wastes in a hazard-free manner.

Keywords: E-Waste, environmental impacts, health impact, hotspot, recycling

1. INTRODUCTION

Rapid population growth with rapid discarded product due to the increased access of modern technology with increased purchasing power resulted the generation of electronic waste (e-waste).Also the production of electrical and electronic equipment is increasing worldwide and the life span of some of the equipment is very short. For example computers in the early 1980s were used on average for about ten years but their life span has since reduced to an average of about three years. Mobile phones too become outdated and are replaced on average after about two years (Bogue, 2001). As a consequence of the increasing market expansion in electrical and electronic equipments and their short life span, the waste stream of these products, commonly called "e-waste", is fast growing. This is a significant problem that e-waste contain different toxic materials which are hazardous, and are consequently a threat to the environment and to the human health (Sinha *et al.*, 2007). More than 1,000 hazardous and non-hazardous components like ferrous material (38%), non-ferrous material (28%), plastic (19%), glass (4%) and others (including wood, rubber, ceramic) (11%) contain in the electric and electronic waste (Wath *et al.*, 2011). Some heavy metals like lead, mercury, cadmium, chromium (VI), halogenated constituents (e.g., CFCs), polychlorinated biphenyls, brominated flame retardants (BFRs) can also be found as a substance in those compounds. All these may react as catalyst for the formation of dioxins (DEFRA, 2004) and in turn act as a harmful ingredient for both environment and human health (Wath *et al.,* 2011). According to BEMMA (Bangladesh Electric Manufacturer and Merchandiser Association); Bangladesh consumes around 3.2 million tones of electronic products each year (ESDO, 2010). Of this amount, only 20 to 30 percent each recycled and the rest is released in to landfills, rivers, ponds, drains, lakes, channels and open spaces which are very hazardous. Presently, there is no specific law or ordinance for e-waste management and recycling in Bangladesh. Also there is no formal plant to recycle e-waste in a hazard-free manner. Most of these electronic products are recycled by the informal sector located mainly in Dhaka and Chittagong (Ahmed, 2010). When handled improperly, e-waste presents significant human health and environmental risks due to the toxicity of materials used in many electronic products (Environment Canada, 2004; Dayaneni & Doucette, 2005; Leung *et al.,* 2006 ; Huo *et al.,* 2007; Wong *et al.,* 2007; Fu *et al.,* 2008).

The lack of available information regarding the handling of the expired electronics appliances and improper monitoring system relating to dumping does not raise the human health issue only. It can also contaminate the agricultural soil contents with the reduction of annual crop production or can deteriorate the surface and subsurface water ways. Atmospheric pollution due to burning and dismantling activities seems to be the main cause of occupational and secondary exposure (Sepúlveda, 2010). Informal sector e-waste activities are also a crucial source of environment-to food-chain contamination, as contaminants may accumulate in agricultural lands and be available for uptake by grazing livestock. In addition, most chemicals of concern have a slow metabolic rate in animals, and may bio accumulate in tissues and be excreted in edible products such as eggs and milk. E-waste-related toxic effects can be exacerbated throughout a person's lifetime and across generations (Frazzoli *et al.,* 2010).

As electronic products become increasingly part of our daily lives and of our waste stream, it is important to consider the potential health risks associated with the materials contained in these products. The following are some of the potentially hazardous materials contained in various electronic products or associated either with electronics manufacturing or e-waste processing. Antimony (found in CRTs, printed circuit boards, etc.) is very hazardous in event of ingestion, hazardous in event of skin and eye contact, and inhalation. It also causes damage to the blood, kidneys, lungs nervous system, liver and mucous membranes (MSDS, 2005). Soluble inorganic arsenic (for making transistors) is acutely toxic and intake of inorganic arsenic over a long period can lead to chronic arsenic poisoning. Effects, which can take years to develop, include skin lesions, peripheral neuropathy, gastro- intestinal symptoms, diabetes, renal system effects, cardiovascular disease and cancer (WHO, 2010a). Short-term exposure of barium (found in front panel of CRTs) causes muscle weakness and damage to heart, liver and spleen. It also produces brain swelling after short exposure (Osuagwu & Ikerionwu, 2010). Cadmium (found in Chip resistors and semiconductors) has toxic, irreversible effects on human health and accumulates in kidney and liver (op. cit.). Has toxic effects on the kidney, the skeletal system and the respiratory system, and is classified as a human carcinogen (WHO, 2010b). Chlorofluorocarbon, CFCs (found in older fridges and coolers) destroy the ozone layer and is a potent greenhouse gas. Direct exposure can cause unconsciousness, shortness of breath and irregular heartbeat. Can also cause confusion, drowsiness, coughing, sore throat, difficulty in breathing, and eye redness and pain. Direct skin contact with some types of CFCs can cause frostbite or dry skin (USNLM, 2011). Cobalt (found in Rechargeable batteries and coatings for hard disk drives) is hazardous in case of inhalation and ingestion, and is an irritant of the skin. Has carcinogenic effects and is toxic to lungs. Repeated or prolonged exposure can produce target organs damage (MSDS, 2005). Copper (in conductor) is very hazardous in case of ingestion, in contact with the eyes and when inhaled. An irritant of the skin and toxic to lungs and mucous membranes. Repeated or prolonged exposure can produce target organs damage (MSDS, 2005). Lead (found in solder of printed circuit boards, glass panels and gaskets in computer monitors) causes damage to central and peripheral nervous systems, blood systems and kidneys, and affects the brain development of children (Osuagwu & Ikerionwu, 2010). A cumulative toxicant that affects multiple body systems, including the neurological, hematological, gastrointestinal, cardio vascular and renal systems (WHO, 2010c). Elemental and methyl-mercury (found in relays, switches and printed circuit boards) are toxic to the central and peripheral nervous system. Inhalation of mercury vapour can produce harmful effects on the nervous, digestive and immune systems, lungs and kidneys, and may be fatal. The inorganic salts of mercury are corrosive to the skin, eyes and gastrointestinal tract, and may induce kidney toxicity if ingested (WHO, 2007). Nickel (found in rechargeable batteries) is slightly hazardous in case of skin contact, ingestion and inhalation. May be toxic to kidneys, lungs, liver and upper respiratory tract. Also has a carcinogenic effect (MSDS, 2005). This study aims at preparing a comprehensive inventory of the level of e-waste management practice in the perspective of large cities in Bangladesh and finally a general physical model was proposed in consultation with the relevant stakeholders for its long-term sustainability.

2. METHODOLOGY

Recycling is clearly a waste-management strategy, but it can also be seen as one current example of implementing the concept of industrial ecology, whereas in a natural ecosystem there are no wastes but only products (Frosch & Gallopoulos, 1989; McDonough & Braungart, 2002). Some of the major stakeholders in the life cycle of e-waste include producers/manufacturers, retailers (businesses/government/others), consumers (individual households/businesses/government/others), traders, exporters and importers, scrap dealers, disassemblers/dismantlers, smelters and recyclers (UNEP, DTIE, 2007). The recyclers are often specialized in recovering specific materials. The e-waste recycling sector in developing countries is largely unregulated and the process of recovering valuable materials takes place in small workshops using simple recycling methods. The practices used in developing countries often exacerbate pollution by creating hazardous chemicals and additional pollution.

This study used both primary and secondary information sources. In order to have an idea about the current status of e-waste recycling in the informal sector, existing literature were reviewed. In addition to this a primary survey has been conducted in Dhaka and Chittagong (Figure 1) with structured questionnaires to collect information from recycles shop owners and workers regarding the recycling process. However the scope of the study was limited to Dhaka and Chittagong. The interviewers surveyed two shops at each spot and closely observed rest of the shops to infer on the entire spot. Although no definite official data exist on how much waste is generated in Bangladesh or how much is disposed of, there are estimations based on independent studies conducted by the NGOs or government agencies.

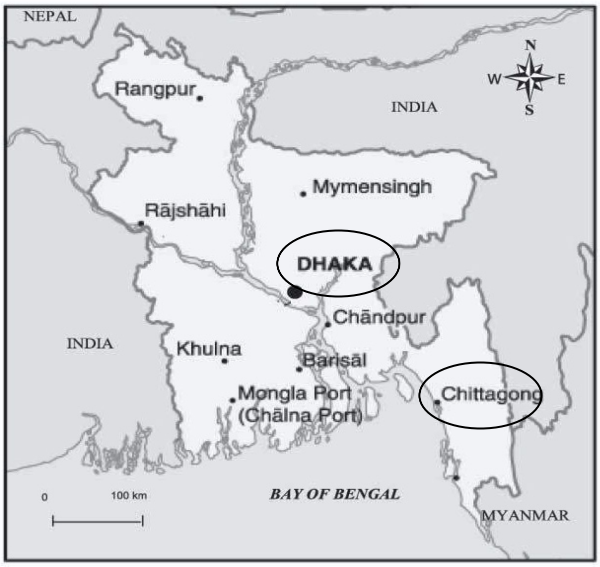
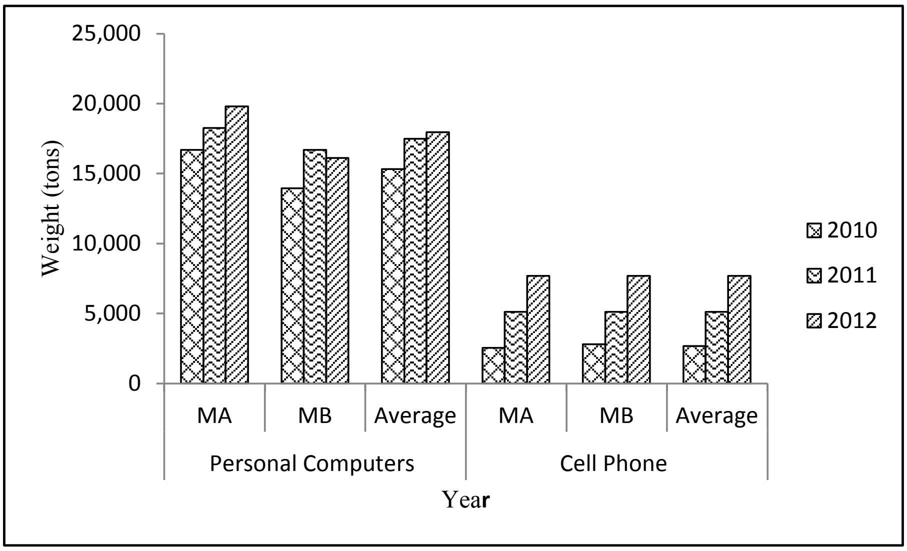


Figure 1: Location of the Survey area

3. RESULTS AND DISCUSSION

3.1 Quantity of E-Waste

According to an estimate, more than 500 thousand computers were in use in 2004 and this number has been growing at 11.4 percent annually (Hossain, 2004). Even if the figure of 500 thousand were taken as the base line, that many PCs would contain approximately 15,323tonnes of waste (@ 27.2 kg/PC for 5 year obsolescence) in 2010 contains deadly plastics, lead, mercury etc. The quantity of e-waste (PC and Cell phone) to be generated has been estimated by following two methods suggested in (Sinha et al., 2007). The first method, Market Supply Method A (MA) assumes that the average life time of an electronic product is approximately five years and after that these are discarded and come to the waste stream. The second method, Market Supply Method B (MB) assumes that all the products are not disposed at the same time, rather they are disposed in varying quantities over successive years. Here weighted average method is used to show the product disposal trend. For PCs the growth rate is considered to be 11.4 percent (Hossain, 2004) and for cell phones a 100% growth rate is considered annually (Pervez et al., 2007). The quantity of e-waste to be generated from these two types of electronic products is shown in Figure 2. According to recent study and available information, approximately (50,000) fifty thousand children are involved in the informal e-waste collection and recycling process, amongst them about 40% are involved in ship breaking yards (ESDO, 2012).



(Source: Ahmed, 2011)

Figure 2: Estimation of PC and Cell Phone Waste in Dhaka City Area

Note: 1.Weight of PCs is derived 27.2 kg/PC
2. Weight of Cell Phones derived 0.079 kg/Cell Phone

Estimated e-waste generated in Bangladesh each year can be summarized like this (Table 1):

Table 1: Generated E-Waste from individual sectors per year in Bangladesh

Source of E-Waste	**Estimated E-Waste**
Ship Breaking Yards	2.5 million metric ton/yr (2500000 metric ton/yr)
Television Sets	0.182 million metric ton/yr (181896 metric ton/yr)
Computers	0.0084 million metric ton/yr (25244.24 metric ton/30yrs)
Mobile Phones	0.0006 million metric ton/yr (6233.04 metric ton/10yrs)
CFL Bulbs	0.0001 million metric ton/yr (566.90 metric ton/6yrs)
Mercury Bulbs	0.0018 million metric ton/yr (1861.32 metric ton/10yrs)
Thermometers	0.0002 million metric ton/yr (8513.59 metric ton/50yrs)
Other Medical & Dental Waste.	0.009 million metric ton/yr (93478.25 metric ton/10yrs)
Total	2.702 million metric tons/yr

(Source: ESDO, 2012)

3.2 E-waste Recycling Areas

E-waste recycling involves the disassembly and destruction of the equipment to recover new materials (Cui and Zhang, 2008). Recycling can recover 95% of the useful materials from a computer and 45% of materials from cathode ray tube monitors (Ladou and Lovegrove, 2008). Modern techniques can recover high-Pb glass from discarded CRT with minimal environmental impact (Andreola *et al.,* 2007). Any ecological benefits of recycling are more than offset if the waste has to be transported long distances due to the negative environmental effects of fossil fuel combustion (Barba-Gutierrez *et al.,* 2008). However, recycling always has a lower ecological impact than landfilling of incinerated E-waste (Hischier *et al.,* 2005).

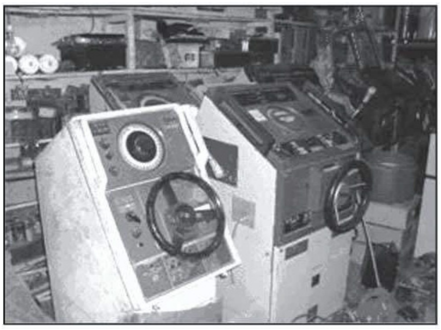


Figure 3: E-Waste Recycling in Dhaka Figure 4: E-Waste Recycling in Chittagong

There are different areas in Dhaka & Chittagong city (Figures 3& 4) that handles second hand electronic products. Among them following are the key areas dealing with e- waste recycling. In Dhaka the areas with the most concentrated disposal and storage of e -waste are in Islampur, Kamrangirchar, Gingira, Mirpur and Mohammadpur (ESDO, 2010). Nimtali, in chankhar pool, is the largest computer vangari spot in Dhaka. There are few Vangari spots in other places beside Nimtali of Dhaka thes are DolaiKhal, Elephant road, Kazipara, Shewrapara, Gulshan 1, 2. There are different areas in Chittagong that handles second hand electronic products. Among them following are the key areas dealing with e-waste recycling: CDA market, Coxy market, Ice Factory Road, Vatiary, Kadamtali (Ahmed, 2011).

3.3 Hotspot Characteristics

Most of the shops in different recycling markets of Dhaka opened during the last 3-4 years and they are handling mainly PC and related materials. Thus PC is the main source of e-waste in Dhaka. The main source of e-waste in Chittagong is the ship breakage industry. Almost 95% of the e-waste generated in Chittagong is from this particular sector. Thus it can be commented that, without this particular ship breakage industry, Chittagong would have been less burdened with toxic e-waste problem. Workers in the recycle shops in Dhaka are receiving monthly wages of approximately BDT 3000 working for 8-12 hours daily. This is lower than the wages earned by day laborers. Thus the recycle workers are getting less payment for hazardous job. The survey on the recycle shop workers revealed that, on an average they earn BDT 3000 monthly working 12 hours a day. Compared to other professions, the workers engaged in e-waste recycling are getting lower wages. A day labourer in this city earns minimum BDT 200 for working 8 to 9 hours a day. Hence recycling as a profession is less financially rewarding in spite of being hazardous. Most of these shops in Dhaka are using pliers, hammer, chisel, screwdriver as a tool to break those things. From most of the items they could not recover important parts which they could have if used modern technology. This inefficiency is resulting into higher quantity of wastage and scraps. Although the recycle shop owners in Chittagong commented during the survey that, they employ skilled people to recycle the electronic products. However, the discharging of scraps up to 50% of the purchased quantity suggests that, their recycling process is elementary and less efficient. They have reported only the use of hammers, screw-drivers and chisels in recycling, which indicates their low level of operational efficiency. Workers and the owners in both cities do not think that recycling electronic products are hazardous. The lack of visibility of toxic material contained in e-waste by naked eyes making them believes that these are toxic free. There is huge knowledge gap among the shop owners and workers. The owners of the recycling shops in Chittagong are selling their electronic products at 20 to 100 percent markup. Thus the shop owners are extracting a higher profit margin. Also their net gain becomes even higher because of the opportunity to get at a cheap rate. From survey, in Dhaka PC parts like mother boards are exported to China and India. After repairing these are again imported in Bangladesh. There are illegal import and export channels of e-products in Bangladesh. During the survey most of the shop owners reported that they are in the business for 3 to 4 years. Thus the growth of this e-product recycling is relatively new in Dhaka (Ahmed, 2011).

3.4 Recycling Flow

The recycling flow of the informal sector in Dhaka & Chittagong is shown in Figures 5 & 6. Vangari shops buy pc form various organization through auction. They also buy from hawkers, personal users, retail shops (old parts) and internal buying among the vangari shops.

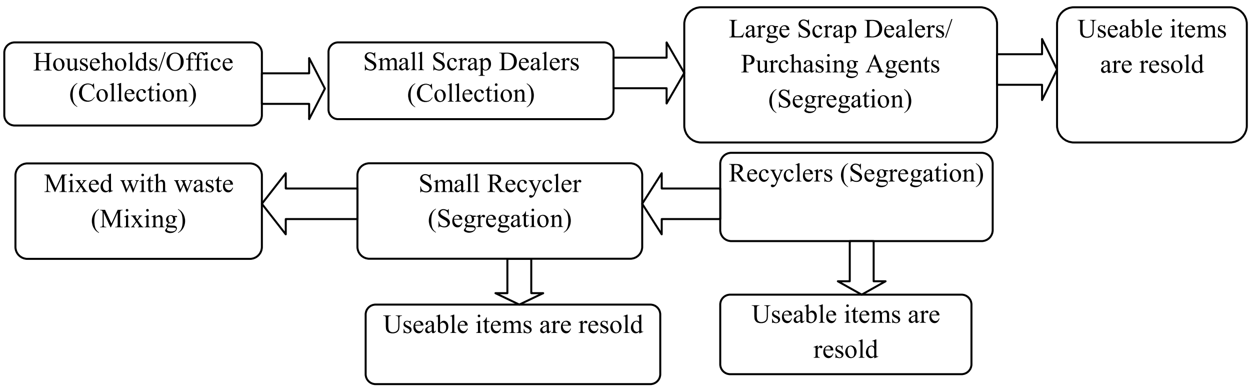


Figure 5: Informal sector recycling process in Dhaka (Ahmed, 2011)

According to the shop owners there are 200-250 purchasing agents of these types of products who bring pc parts as wastage to them. After purchasing a waste product they first run a check to see whether the product is functioning or not. If the product is functioning then they sell it to a purchaser who looks for second hand parts. Otherwise they break the product into pieces to separate iron, lead, copper, silver, plastic etc. and sell this to a purchaser of this thinks. They disassemble these products without any protection which can be injurious to their health and surrounding environment.

It is estimated that 120,000 urban poor from the informal sector are involved in the recycling trade chain in Dhaka city. 15% of the total waste generated in Dhaka (mainly inorganic) equates to 475 tons recycled daily. Of this amount, only 20% to 35% is recycled, while the remainder is disposed of in landfills, rivers, ponds, drains, lakes and open spaces (ESDO 2010).

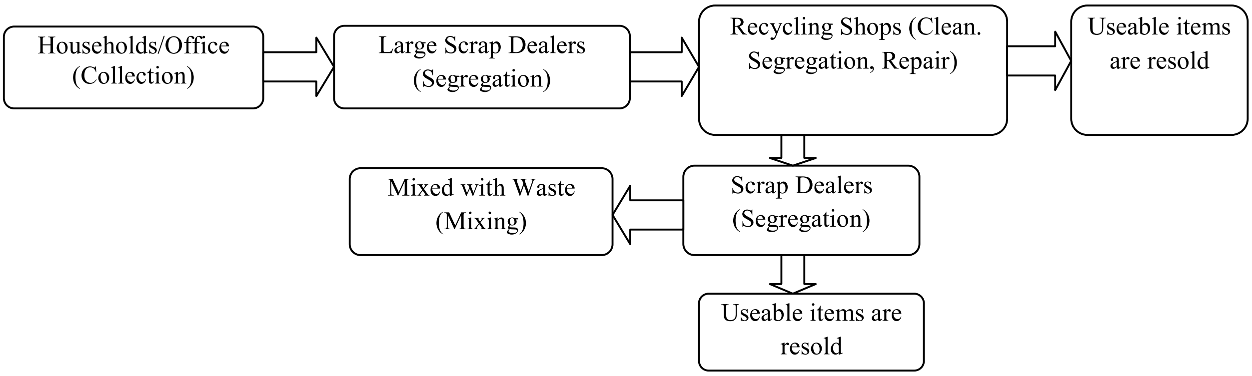


Figure 6: Informal sector recycling process in Chittagong (Ahmed, 2011)

4. PROPOSED MODEL FOR BANGLADESH

Currently, there are no proper waste management guidelines or regulations in place. Reuse of e-equipment is a common practice in Bangladesh. All the recycling is being carried out by the informal sector only in Dhaka and Chittagong. In the informal sector, the process of recycling in Bangladesh has the potential to be hazardous to the recycler's health and environment. Also due to their lack of knowledge, the recovery yield of the precious metals is very poor and, thereby, substantial percentage of the metals like copper, gold, silver, and other precious metals (palladium, tantalum, platinum, etc.) are lost. So this process is also unable to provide sufficient support to economy of Bangladesh. Here a model is recommended for recycling of e-waste (Figure 7), which may ensure proper collection and recycling of its various parts. It would also be favourable for health, environment and economy.

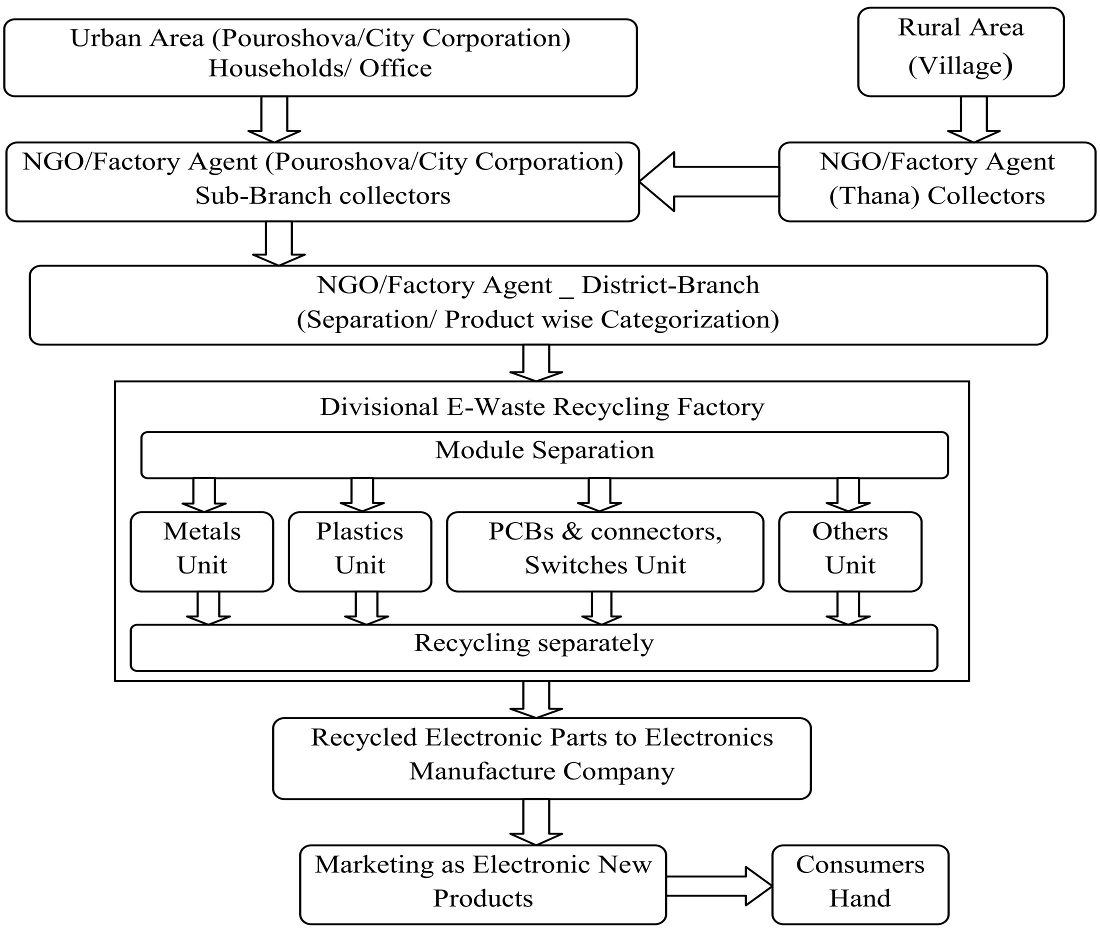


Figure 7: Proposed model for Sustainable Recycling of E-Wastes

In this process, recycling factory should have one divisional factory in every district. NGO/ Recycling Factory agent of Pouroshova/City Corporation in urban area collect electronic waste from household/office. In rural area, thana collectors collect the waste and transfer these to pouroshova/city corporation. District branch collectors collect the waste and separate it according to types such as parts of computer, circuits, cables etc. The parts of different electronic equipment then send to the divisional recycling factory. In divisional recycling factory, workers of different units separate the module and then recycle separately. After proper recycling process, the recycled parts send to the manufacturers. Manufacture company produce electronic new products by different process such as CRT monitors are converted into low cost television sets and video game monitors etc. Refurbishers engage in activities that bring non-working rubbish electronics back into working condition. The total volume of material derived from rubbish electronics moved into new rounds of production. New products go to the consumer hand and reuse it.

6. CONCLUSIONS

This paper has provided some qualitative and quantitative information on e-waste recycling sector in Bangladesh. Estimated e-waste was found the highest (2.5 million metric ton/yr) in ship breaking yards followed by 0.182 million metric ton/yr for television sets and so on in individual sectors in Bangladesh. All the recycling is being carried out by the informal sector only in Dhaka and Chittagong in Bangladesh. It is estimated that 120,000 urban poor from the informal sector are involved in the recycling trade chain in Dhaka city. To ensure better human health and safety of workers involving in the process of waste disposal, effective e-waste recycling management system is needed which is sustainable. The formulated general physical model suggests that the large number of waste wholesale shops in the urban and rural area should be properly adjusted with their upper and lower chains in order to improve the overall reuse and recycling scheme. Furthermore, a comprehensive training program on personal hygiene was deemed imperative for the workers in all recycling schemes. In acclaimed management process, generated EW is collected by NGO who are the affiliates of existing social system. They can go from door to door and can attract the people about the proposed model. This model is selected for divisional recycling and so burden is released from the capital city. This study tried only to unfold a theoretical model for better e-waste recycling process in Bangladesh. To investigate the possibility of this model, a complete empirical study is necessary. This study will also prepare the platform for additional study and exploration of the e-waste recycling.

REFERENCES

Ahmed, S.U. (2010). “E-Waste: a growing concern for ICT-based growth and development- a first cut analysis.” Research Paper Series 60007. Development Research Network, Dhaka.

Ahmed, S.U. (2011). Informal Sector E-waste Recycling Practices in Bangladesh. Dhaka, Bangladesh.

Andreola F., Barbieri L., Corradi A., Ferrari AM., Lancellotti I., Neri P. (2007). “Recycling of EOL CRT glass into ceramic glaze formulations and its environmental impact by LCA approach.” International Journal of Life Cycle Assessment, Vol. 12, 448–54.

Barba-Gutierrez Y., Adenso-Diaz B., Hopp M. (2008). “An analysis of some environmental consequences of European electrical and electronic waste regulation.” Res Conserv Recycl 2008, Vol. 52, 481–95.

Bogue, D. (2001). “Electronic scrap: a growing resource.” MBM, 21-24.

Cui, J.R. and Zhang, L.F. (2008). “Metallurgical recovery of metals from electronic waste: a review.” Journal of Hazard Mater 2008, Vol. 158, 228–56.

Dayaneni, G., and Doucette, J. (2005). “System Error—Toxic Tech Poisoning People and Planet (San Jose, CA: Silicon Valley Toxics Coalition).”
http://etoxics.org/site/DocServer/SVTCcampusFINAL.pdf?docID=121 (accessed on 14 September 2013).

Department for Environment, Food and Rural Affairs (DEFRA) Report (2004). March 2004 <www.defra.gov.uk>.

Environment Canada (2004). “Screening Level Human Health and Ecological Risk Assessment for Generic E-waste Processing Facility.” Ottawa, ON: National Office of Pollution Prevention, Environment Canada.

Environment and Social Development Organization-ESDO (2010). Study Report: E-waste: Bangladesh Situation. Dhaka, Bangladesh.

Environment and Social Development Organization-ESDO (2012). Guidelines for E-Waste Management in Bangladesh. Dhaka, Bangladesh.

Frazzoli, C., Orisakwe, O.E., Dragone, R., Mantovani, A. (2010). “Diagnostic health risk assessment of electronic waste on the general population on developing countries’ scenarios.” Environmental Impact Assessment Review, Vol. 30 (6), 388–399.

Frosch, R. & Gallopoulos, N. (1989). “Strategies for manufacturing.” Sci. Am., Vol. 261, 144–152.

Fu, J. J., Zhou, Q. F. J., Liu, M., Liu, W., Wang, T., Zhang, Q. H., and Jiang, G. B. (2008). “High levels of heavy metals in rice (Oryza sativa L.) from a typical E-waste recycling area in southeast China and its potential risk to human health Chemosphere.” Vol. 71(7), 1269–1275.

Hischier R, Wager P, Gauglhofer J. (2005). “Does WEEE recycling make sense from an environmental perspective? The environmental impacts of the Swiss take-back and recycling systems for waste electrical and electronic equipment (WEEE).” Environtal Impact Assessment Review 2005, Vol. 25, 525–39.

Hossain, A. (2004). Country Presentation: Bangladesh. First Regional Conference on Follow up to the 1st Phase and Preparation for the 2nd Phase of the WSIS, Bangkok.

Huo, X., Peng, L., Xu, X. J., Zheng, L., Qiu, K. B. Z., Zhang, L. B., Han, D., and Piao, Z. X. (2007). “Elevated blood lead levels of children in Guiyu, an electronic waste recycling town in China.” Environmental Health Perspectives, Vol. 115(7), 1113–1117.

Ladou J. and Lovegrove S. (2008). “Export of electronics equipment waste.” International Journal of Occupational & Environtal Health 2008, Vol. 14, 1-10.

Leung, A., Cai, Z. W., and Wong, M. H. (2006). “Environmental contamination from electronic waste recycling at Guiyu, southeast China.” Journal of Material Cycles and Waste Management, Vol. 8, 21–33.

Material Safety Data Sheet (MSDS) (2005). “Material Safety Data Sheet Listing.” http://www.sciencelab.com/msdsList. php (accessed on 25 September 2013).

McDonough, W. & Braungart, M. (2002). “Cradle to cradle: remaking the way we make things New York.” NY: North Point Press.

Osuagwu, O.E, Ikerionwu, C (2010). “E-cycling e-waste: The way forward for Nigeria IT and electro-mechanical industry.” International Journal of Academic Research, Vol. 2 (1), 142–149.

Pervez, M., Hossain, M. and Bari, A. (2007). “Nonfunctional Cell Phone- a hazardous waste.” The New Nation.

Sepúlveda, A. (2010). “A review of the environmental fate and effects of hazardous substances released from electrical and electronic equipments during recycling: Examples from China and India.” Environmental Impact Assessment Review, Vol. 30 (1), 28–41.

Sinha, S., Wankhade, K. and Khetriwal, D. (2007). “Mumbai Choking on a E-Waste in Mumbai.” Toxics Link.

UNEP, Division of Technology, Industry and Economics (DTIE), International Environmental Technology Centre (2007). “E-waste: Volume 1: Inventory Assessment Manual (Osaka/Shiga).” http://www.unep.or.jp/ietc/Publications/spc/EWasteManual_Vol1. pdf (accessed on 8 Oct. 2013).

US. National Library of Medicine. n.d. (USNLM) (2011). “Tox Town: Chlorofluorocarbons (CFCs).” http://toxtown.nlm.nih.gov/text_version/chemicals.php? Id=9 (accessed on 8 Oct. 2013).

Wath, S. B., Dutt, P. S., & Chakrabarti, T. (2011). “E-Waste scenario in India, its management and implications.” Environmental Monitoring and Assessment, Vol. 172, 249-262.

Wong, M. H., Wu, S. C., Deng, W. J., Yu, X. Z., Luo, Q., Leung, A. O. W. Wong, C. S. C., Luksemburg, W. J., and Wong, A. S. (2007). “Export of toxic chemicals—a review of the case of uncontrolled electronic-waste recycling.” Environmental Pollution, Vol. 149(2), 131–140.

World Health Organisation (WHO) (2007). “Exposure to mercury: A major public health concern.” http://www.who.int/ipcs/features/mercury. pdf (accessed on 5 September 2013).

World Health Organization (WHO) (2010a). “Exposure to arsenic: A major public health concern.” http://www.who.int/ipcs/features/arsenic. pdf (accessed on 5 September 2013).

World Health Organisation (WHO) (2010b). “Exposure to cadmium: A major public health concern.” http://www.who.int/ipcs/features/cadmium. pdf (accessed on 5 September 2013).

World Health Organisation (WHO) (2010c). “Exposure to lead: A major public health concern.” http://www.who.int/ipcs/features/lead. pdf (accessed on 5 September 2013).

ENGINEERING BEHAVIOUR OF SOLID WASTE IMPROVED BY SAND

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ABSTRACT

A lot of wastes are produced everyday through human activities, in which solid waste is most. These wastes are dumped near the suburb of a city. After 20 to 30 years later these wastes or landfill became a part of soil. Due to the filling of low land with solid wastes, at the end of filling, a considerable area of land will be available that can be used for urban development. For the increasing population, the human traffic diverted from main city to its suburb portion. The newly developed land with decomposed wastes can used for construction of multistory buildings for living, shopping or for business purpose etc. However, it is very important to know the main engineering properties of that soil before going to urban development in that area. In this work it is figure out the main engineering properties of solid waste and also tried to improve the basic soil properties with mixing of sand of various proportions as an admixture.

Keywords: Solid waste, landfill, admixture, consistency, shear strength, consolidation.

1. INTRODUCTION

Every year millions of tons of municipal solid wastes are produced in Bangladesh and worldwide, and while a growing amount is recycled and recovered. Waste materials are disposed in landfills. At the same time, the strict environmental criteria for the donation of landfills, the extensive urbanization and the unwillingness of people to live near a landfill reduce significantly the locations where landfills could be constructed and operated. This situation applies great pressure at existing landfills to place more waste than was specified in the original design. Such decisions are often based on economical and political considerations, and engineers need to find innovative ways to accommodate more waste in existing landfills. As long as other attractive methods for the disposal of large volumes of waste material do not exist, the pressure for placement of more waste in existing landfills will increase. Safe placement of more waste in existing landfills requires sound engineering analyses. For the reliable performance of such engineering analyses, reasonable characterization of the mechanical response of the waste material is required. Also it is very vial to understand the physical properties of that sample. To identify the certain soil property of that solid waste some experiments are essential. For that purpose, to know the basic engineering properties of solid waste consistency, shear strength, consolidation tests are very effective. Here, also some kind of admixture could be mixed with the sample to show the variation of properties. If any kind of major construction is carried out on that particular type of soil, then it is very important to know the basic engineering properties of that soil. It could be beneficial for the future, if the properties of that landfill is known. Also, sand could be used as an admixture to modify the engineering properties of the solid waste.

2. MATERIALS AND METHODOLOGY

2.1 Materials

Solid wastes decomposed in landfill are considered as the experimental material ans local sand is considered as admixture for improving the enginnering properties. Sand is pure silica. And free from solid waste, silt, organic matter, shells and salts. According to the size of grains, the sand is classified as fine, coarse and gravelly. The sand passing through a screen with clear openings of 1.5875 mm is known as the fine sand. The sand passing through a screen with clear opening of 3.175 mm is known as the coarse sand. The sand passing through a screen with clear openings of 7.62 mm is known as the gravelly sand. Different proportion of this sand is used in experiment. The properties of decomposed solid wastes and admixture are presented in Table 1. The grain size distribution of decomposed solid wastes and admixture sand are presented in Fig. 1 and 2.

Table 1: Properties of the admixture (Local sand)

Test properties	Specific gravity	Moisture content (%)	Liquid limit (%)
Solid waste	2.33	15.94	44
Sand (Admixture)	2.52	16.03	50

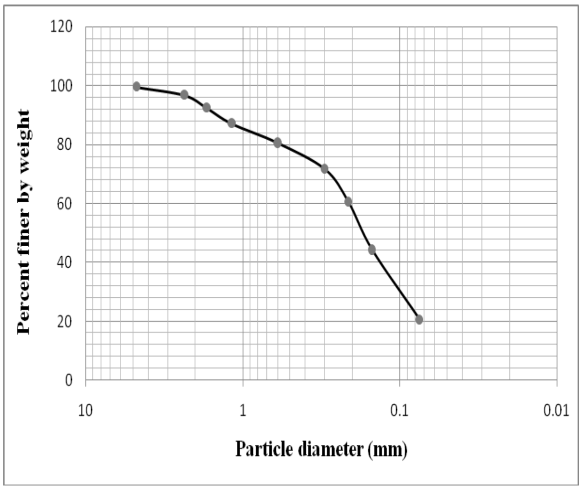

Fig. 1: Grain size distribution curve of sample.

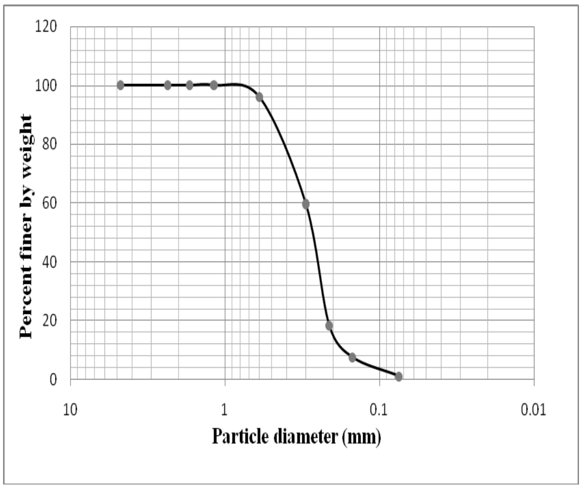

Fig. 2: Grain size distribution curve of sand.

These two figures indicate the nature of the sample and the admixture. To identify the sample this grain size distribution curve is very vital. Actually it is very essential for soil to define its further new identification.

2.2 Methodology

To understand the basic character of the soil, some soil tests are to be performed. Atterberg limit test (consistency test), shear strength test and consolidation test are performed to determine the basic properties of solid waste. Also, sand with different percentages was mixed to observe the variation of the properties.

3. EXPERIMENT AND RESULTS

3.1 Atterberg Limit Test (Consistency Test)

The consistency of a fine grained soil is the physical state in which it exists. It is used to denote the degree of firmness of soil. Consistency of a soil is indicated by such terms as soft, firm or hard. In 1911, a Swedish agriculture engineer Atterberg mentioned that a fine-grained soil can exists in four states, namely liquid, plastic, semi-solid or solid state. The water content at which the soil changes from one state to another are known as consistency limits or Atterberg limits. Consistency limits are very important index properties of fine-grained soils. The following figures (Fig. 3 to 5) illustrates the main feature of the experiment.

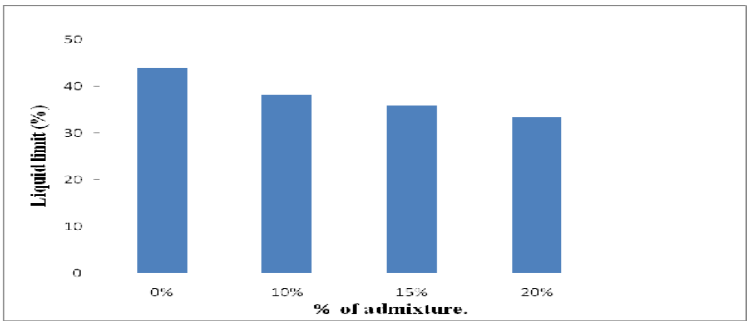

Fig. 3: Variation of LL with the increment of sand

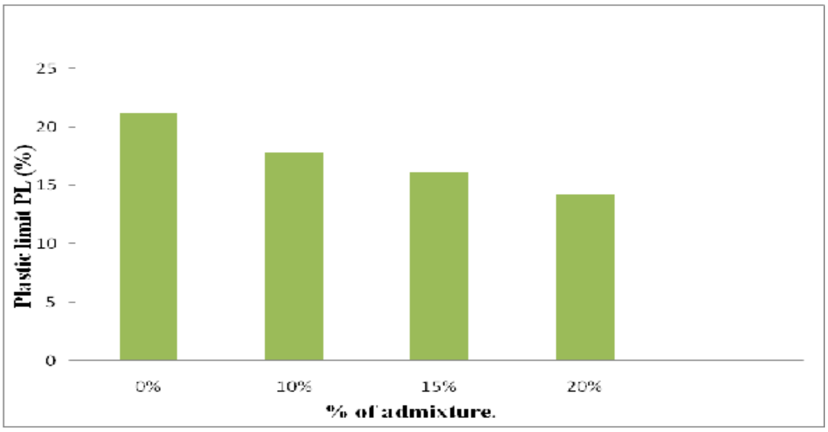

Fig. 4: Variation of PL with the increment of sand

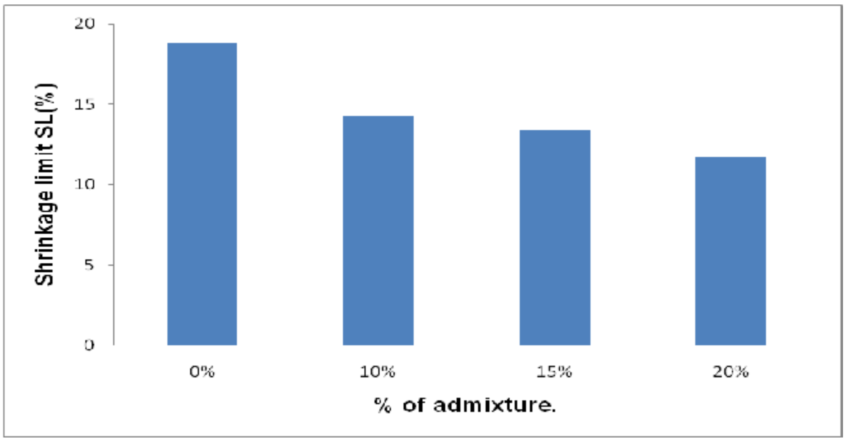

Fig. 5: Variation of SL with the increment of sand

From the above figure, it clearly visible that Atterberg limits are decreased with the increment of sand. For the raw sample the LL about 45%, PL 22% and SL about 18%. After mixing the sand admixture the limits were decreased and also went in decrease with the increased proportion of sand admixture.

3.2 Shear Strength Parameters

The shear strength of a soil mass is the internal resistance per unit area that the soil mass can offer to resist failure and sliding along any plane inside it. One must have understand the nature of shearing resistance in order to analyze soil stability problems such as bearing capacity, slope stability, and lateral pressure on earth retaining structures. (Das, B. M. (1999), "*Principles of Geotechnical Engineering*," Fourth Edition.) When soil is loaded, stresses are induced in it. When the shearing stresses reaches a limiting value, shear deformation takes place, leading to the failure of the soil mass. The failure may be in the form of sinking of footing, or movement of a wedge of soil behind a retaining wall forcing it to move out, or slide in an earth embankment. The shear strength of soil is the resistance to deformation by continuous shear displacement of soil particles or on masses upon the action of a shear stress. The failure condition for a soil may be expressed in terms of limiting shear stress called shear strength, or as a function of the principle stresses. Shear strength parameters means cohesion and angle of internal friction. Using direct shear apparatus we can determine these two parameters. These are very vital parameters to identify the soil character. The table below shows the variation of these two parameters.

Table 2: Cohesion and angle of internal friction of raw sample and sample with different percentage of admixture.

Solid waste		Solid waste with 10% sand		Solid waste with 15% sand		Solid waste with 20% sand	
C (kPa)	φ	C (kPa)	φ	C (kPa)	φ	C (kPa)	φ
21	3.7	17.9	5.7	16.7	6.9	15	9.5

The Fig. 6 and 7 are presented to identify the variation due to the sand admixture.

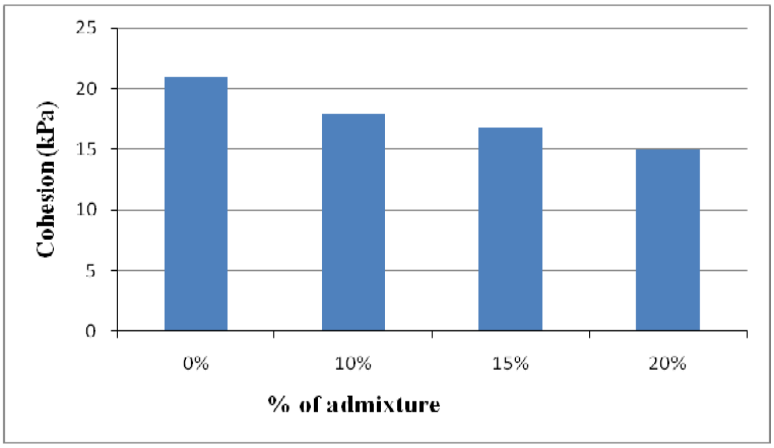

Fig. 6: Variation of cohesion with admixture.

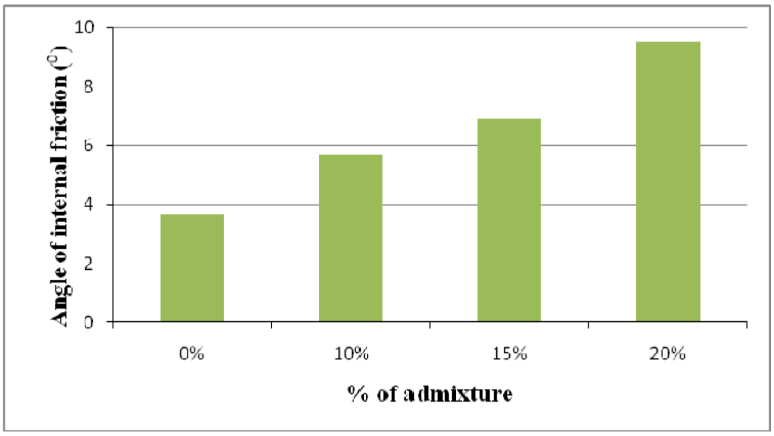

Fig. 7: Variation of angle of internal friction with admixture.

From the above figures, it is clearly visible that, with the increment of sand the cohesion decreases while the angle of internal friction increases. It could be a good notable portion of the shear strength test.

3.3 Consolidation Test

Consolidation is a process by which soils decrease in volume According to Kerl Terzaghi "consolidation is any process which involves decrease in water content of a saturated soil without replacement of water by air. In general it is the process in which reduction in volume takes place by expulsion of water under long term static loads. It occurs when stress is applied to a soil that causes the soil particles to pack together more tightly, therefore reducing its bulk volume. When this occurs in a soil that is saturated with water, water will be squeezed out of the soil. The magnitude of consolidation can be predicted by many different methods. In the Classical Method, developed by Terzaghi, soils are tested with an oedometer tests to determine their compression index. This can be used to predict the amount of consolidation. Knowledge of the rate at which the compression of the soil layer takes place is essential from design considerations. This can be achieved by determining the value of the coefficient of consolidation, Cv. To obtain Cv, it is essential to conduct a routine one-dimensional consolidation test. (Nagaraj, T and Murty, B. R. S. (1985). “Prediction of the Preconsolidation Pressure and Recompression Index of Soils,” Geotechnical Testing Journal, Vol. 8, No. 4, 199-202.)With the obtained time-compression data, and using any one of the several available curve-fitting procedures, Cv can be evaluated. This is a time-consuming process. Also, the fact that many curve-fitting procedures are available in the literature suggests that none of them are completely satisfactory in evaluating Cv and, hence, the large variation in the evaluated values by different procedures. Hence, it is desirable to predict the value of Cv by any correlation equation relating with some simple index property. This will be quite satisfactory, especially so for preliminary assessment purposes. From the present experimental study on remolded soils, it is found that C_v has a better correlation with the shrinkage index, which is the difference between liquid limit and shrinkage limit. The compression index for the calculation of field settlement caused by consolidation can be determined by graphic construction after one obtains the laboratory test results for void ratio and pressure.

Skempton (1944) suggested the following empirical expression for the compression index undisturbed solid waste: Cc = 0.009(LL – 10) (Leonards, G. A., and Altschaeffl, A. G., 1964).

Where, LL = liquid limit

Several other correlations for the compression index are also available. They have been developed by tests on various solid waste.

On the basis of observations on several natural solid waste, Rendon- Herrero (1983) gave the relationship

$$C_c = 0.141 G_s^{1.2}\left(\frac{1+e_0}{G_s}\right)^{2.38}$$ (Punmia, B. C., 2001)

Table 3: Result of consolidation test

Item	Solid waste	10% sand	15% Sand	20% Sand
Co-efficient of consolidation(c_v)	0.014	0.027	0.033	0.046
Compression index(c_c)	0.38	0.284	0.236	.23
Swelling index(c_s)	0.1	0.07	0.06	0.05
Void ratio(e)	0.559	0.604	0.626	0.759
Co-efficient of compressibility(a_v) (cm^2/kg)	0.33	0.25	0.21	0.2
Co-efficient of permeability(k) (cm/min)	$2.96*10^{-6}$	$3.82*10^{-6}$	$4.26*10^{-6}$	$5.23*10^{-6}$

4. DISCUSSION

The main purpose of this work is to identify the engineering behavior of the solid waste. From the first it was the target to find out the main soil parameters of that particular sample. From the grain size analysis, it is likely of clay type sample. After the Atterberg limit test, it shows the variety of like clayey soil. From the direct shear test, cohesion decreases with the increase of the sand admixture and the angle of internal friction increases meanwhile. Which indicates the basic properties of clayey soil, because for pure clay sample angle of internal friction is zero but cohesion exists.

5. CONCLUSIONS

In this work, the soil properties of solid waste and also the variation of the engineering properties of solid waste due to admixture are shown. From the test results it is clearly seen that for solid waste consistency limits, consolidation properties and strength properties are varied when it was mixed with sand. The liquid limit is decreased with an average value of 6%, plastic limit is decreased with an average 5% and shrinkage limit is decreased with an average of 12% with the increment of sand admixture. From consolidation test co-efficient of consolidation, void ratio and permeability are increased with the increment of sand as well as swelling index, compression index are decreased with an average value of 35%. From the direct shear test analysis, cohesion is decreased with an average of 15% as well as angle of internal friction is increased with an average value of 55%

with the increment of sand admixture. From the above discussion we may conclude that the soil consistency limits, consolidation properties and shear strength properties are improved when mixed with sand. The test was performed by adding sand as an admixture. The same work can be performed by adding other admixture (like fly ash, lime, cementing agent etc.).

REFERENCES

Das, B. M. (1999), "*Principles of Geotechnical Engineering*," Fourth Edition.

Houhg, B. K. (1957). *Basic Soils Engineering*, Ronald Press, New York.

Kabbaj, M. (1985). "Aspects Rheologiques des Argiles Naturelles en Consolidation."Ph. D. thesis, Universite Laval, Quebec, Canada.

Leonards, G. A., and Altschaeffl, A. G. (1964). "Compressibility of Clay," *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 90, No. SM5, 133-156.

Nagaraj, T and Murty, B. R. S. (1985). "Prediction of the Preconsolidation Pressure and Recompression Index of Soils," *Geotechnical Testing Journal*, Vol. 8, No. 4, 199-202.

Nishida,Y.(1956)."A Brief Note on Compression Index of Soils," *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 82, No. SM3, 1027-1.

Punmia, B. C. (2001) *Soil Mechanics and Foundations*, Laxmi Publication PVT. LTD, Thirteenth Edition, pp. 24-62.

CONTENTS (Water Resources Engineering)

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STORM WATER MANAGEMENT FOR URBAN AREAS OF BANGLADESH BY ANALYTICAL & MODELLING APPROACH: A CASE STUDY OF CHALNA MUNICIPALITY

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ABSTRACT

Climate change will affect water resources through its impact on the quantity, variability, timing, formation and intensity of precipitation. Improved management of storm water is critical, if addressed inadequately, it will jeopardize progress of poverty reduction targets and sustainable development in all economic, social and environmental dimensions. The purpose of this drainage study is to assess the present drainage situation, identify the future requirements and suggest improvement of the drainage network system to provide the Municipality a area free from water congestion within an acceptable environmental condition. Integrated study of drainage dynamics in consideration of gravity flow for the proposed drainage improvement system is made by developing rainfall-runoff model and analytical computation for the urbanized area. The overall planning processes are conjugated with rigorous study of infrastructure, Digital Elevation Model (DEM), land-use and sattelite image using GIS for a preliminary conceptual understanding of the Municipality system; identification of rivers/khals surrounding the Municipality and collection of data to understand the hydrological response of the Municipality; assessment of effective range of land levels which would be considered for planning process; making of intensive field visit for identification of possible outfalls and drainage routes in verification of the preceding planning processes; planning of drains & zones with scrutinized outfall locations; and finally storm runoff assessment using empirical formula. Modelling approach is used to generate catchment runoffs which are calibrated against flows using empirical formula known as modified rational formula for respective design year.

Keywords: *Storm water management, Strom Runoff, Drainage improvement plan , Changing Climate*

1. INTRODUCTION

Storm water management systems have to be designed to handle the heavy downpours that are expected to increase in intensity and frequency with shifting of storm pattern as a result of climate change. It has been identified that improvement of the drainage system is one of the highest priority needs of the urban area's authority for living environment of its population. The Municipality under this study suffers from drainage congestions and water logging especially during rainy season which creates an unhealthy environmental situation and causes inconvenience to the residents of it including damages to the infrastructure, loss of business and spreading of diseases. It is observed that there is a lack of planned and adequate drainage network system in this Municipality. Existing drains are inadequate in capacities and lack in gradient and also do not reach the suitable outfall. Moreover, those drains are insufficient to deal with the full drainage resulting from rainfall runoff. The objectives of this drainage study is to assess the present drainage situation, identify the future requirements and suggest improvement of the drainage network system to provide the Municipality a area free from water logging/congestion within an acceptable environmental condition in consideration with climate change which will cause extreme hydrological events like precipitation.

2. METHODOLOGY

2.1 General

Drainage system of Chalna Municipality is assessed through a sequence of analytical processes. The drainage system of Municipality and its response to hydrology govern the planning for its storm drainage system. The overall planning processes comprise: collection of survey data of infrastructures/feature, contour, land-use and image for a preliminary conceptual understanding of the Municipality system with a review of reconnaissance and other available reports; identification of rivers/khals surrounding the Municipality and collection of data to understand the hydrological response of the Municipality; assessment of effective range of land levels which have to be considered for planning process; making of an intensive field visit for identification of possible outfalls and drainage routes in verification of the preceding planning processes; planning of drains & zones with identification of outfall locations/ reaches; and finally storm runoff assessment using empirical formula. Integrated study of drainage dynamics in consideration with gravity drainage for the proposed drainage improvement system is made by developing drainage model for the Municipality. Model is applied to assess the extent of improvement of gravity drainage. The approach comprises development of Rainfall-runoff model for hydrological analysis and correlating the proposed drainage system with the existing river model to assess hydraulic performances. After runoff is assessed, drain sections are proposed using Manning's formula considering several design criteria.

2.2 Analytical Approach

2.2.1 Peak Runoff Calculation

The Modified Rational Method is one of the simplest methods for calculation of runoff. It gives reasonably accurate result and widely used method for calculation of runoff for last few decades. In designing primary and secondary drains of Chalna Municipality the Modified Rational Method is practiced. The runoff is calculated by Modified Rational Method as equation (1).

Peak runoff, $Q_P = C_s\, C_r\, I\, A/360$ (1)

Where;, Q = Peak runoff flow rate (m^3/s)
I = rainfall intensity (mm/hr)
C_s = storage coefficient
C_r = runoff coefficient
A = catchment area (hectares)

2.2.2 Time of concentration

Time of concentration (T_c) is generally defined as the longest runoff travel time for contributing flow to reach the outlet or design point, or other point of interest. It is frequently calculated along the longest flow path physically. The time of concentration is the sum of time of entry (T_e) and travel time (T_t). Time of entry is the time taken for runoff from the farthest point in the contributing area to flow over the ground and enter into the drain. Travel time is the time taken for runoff to flow through the drain. The time of entry (T_e) is estimated using Kirpitch Equation with the minimum time of entry set as 4 minutes. The Kirpitch equation (2) is:

$$T_e = 0.019621\, L^{0.77} / S^{0.385} \quad (2)$$

Where, T_e = time of entry in minutes
L = maximum length of overland flow in metre
S = average ground slope

Travel time (T_t) is calculated by dividing the length of drain by the water velocity.The rainfall after evaporation and infiltration accumulates first in the depressions, until these have been reached their capacity and then runoff. To take these effects a storage coefficient is used. The value of the storage coefficient is used 0.7 as Chalna Municipality is central area mixed commercial and housing also residential areas with detached houses. The runoff coefficient represents the ratio between the volume of runoff and the volume of rainfall. Runoff coefficient is used 0.4 for Chalna Municipality as residential areas with detached houses. Figure 1 shows Flow Chart for Drainage Study by Analytical approach.

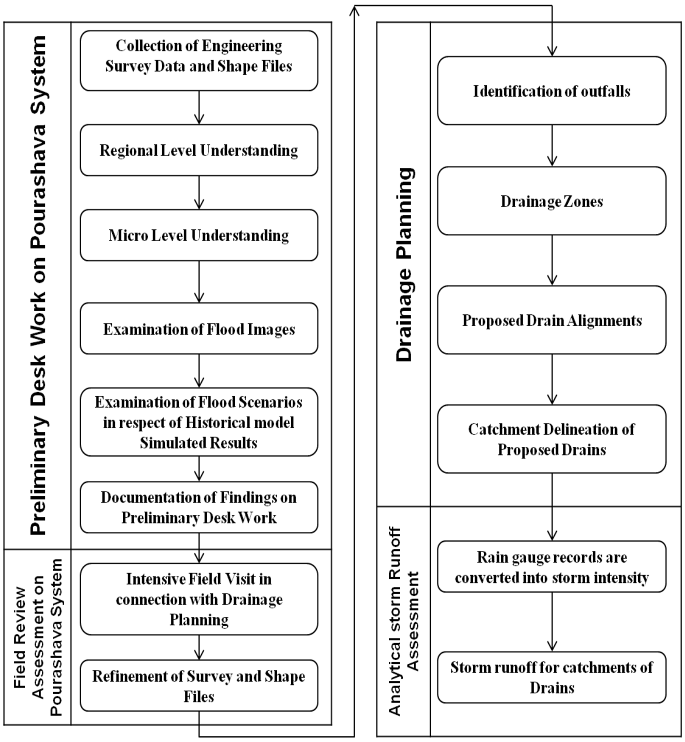

Figure 1: Flow Chart for Drainage Study (Analytical)

2.3 Modelling Approach

Modelling approach is used to generate catchment runoffs. The runoffs are calibrated against flows using modified rational formula. The study includes data collection from primary and secondary sources, analyzing and checking of data, development of hydrological model, reviewing and correlating Municipality drainage system with existing regional models developed by IWM, identification of design year and simulation of the model for the design year, determination of design flows from model simulation, calculation of design parameters from design flows etc. Design years to carry out simulations are determined from the statistical of historical 2-day maximum rainfall data. Long term simulated water levels of Municipality-outfall from the existing regional river model are statistically analyzed to find out water levels for average year. Municipality drainage systems are correlated with the average water level of Outfall River to review and iterate the proposed parameters of planned drainage systems. Figure 2 shows Flow Chart for Drainage Study by modelling approach.

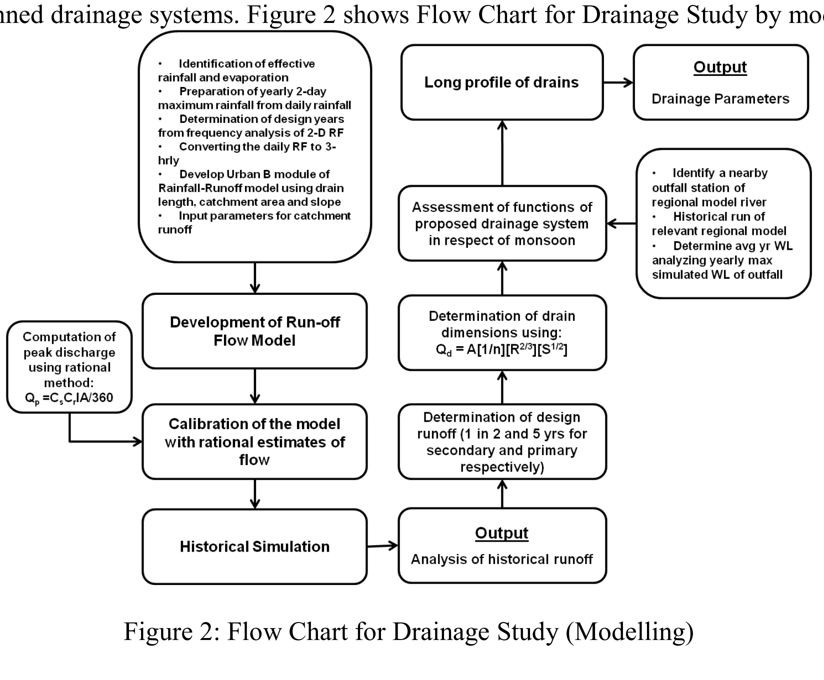

Figure 2: Flow Chart for Drainage Study (Modelling)

2.4 Drain Section Design

2.4.1 Manning's Equation

The Manning's Equation is used for calculation of flow velocity is given below. In determining the dimension of drain, the criterion is that the design discharge (Q_d) should be greater than the peak runoff (Q_p). Manning equation (3) is

$$V = [1/n][R^{2/3}][S^{1/2}] \qquad (3)$$

And Design discharge $Q_d = AV = A[1/n][R^{2/3}][S^{1/2}]$

Where: V = velocity of flow, m3/s
n = Manning's roughness coefficient value
S = Hydraulic gradient, m/m
R = hydraulic radius=A/P, m
A= flow area, m^2

The value of Manning's roughness coefficient 'n' used in the Manning's equation is usually depends on type of drain, for RCC drain, n=0.014 & Earthen drain, n=0.025.

2.4.2 Other Design Criteria

Where possible the minimum velocity in drains should be 0.7 m/s for Tertiary drains and 1 m/s for Secondary drains to ensure that they are self-cleansing. For lined primary drains and outfalls velocity should not exceed 3 m/s and for unlined velocity should not exceed 1.5 m/s. The designed drains should have minimum allowance for freeboard of 200mm for primary drains and 150mm for secondary drains and 100mm for tertiary drains. The recommended longitudinal slopes are 1:500 for tertiary drain, 1:1000 for secondary drain and 1:2000 for primary drain and outfall. The preferred side slope for unlined trapezoidal section for primary drain and outfall is 1:1.5 although in some areas side slopes of 1:2 may be required due to poor ground conditions. For lined channel the longitudinal and side slopes may be steeper than that mentioned.

3. DESCRIPTION OF THE MUNICIPALITY

3.1 Location and Topography

Chalna Municipality is located in Dacope Upazila of Khulna District under Khulna Division. Location of Chalna Municipality and Land-use detail is shown in Figure 3.

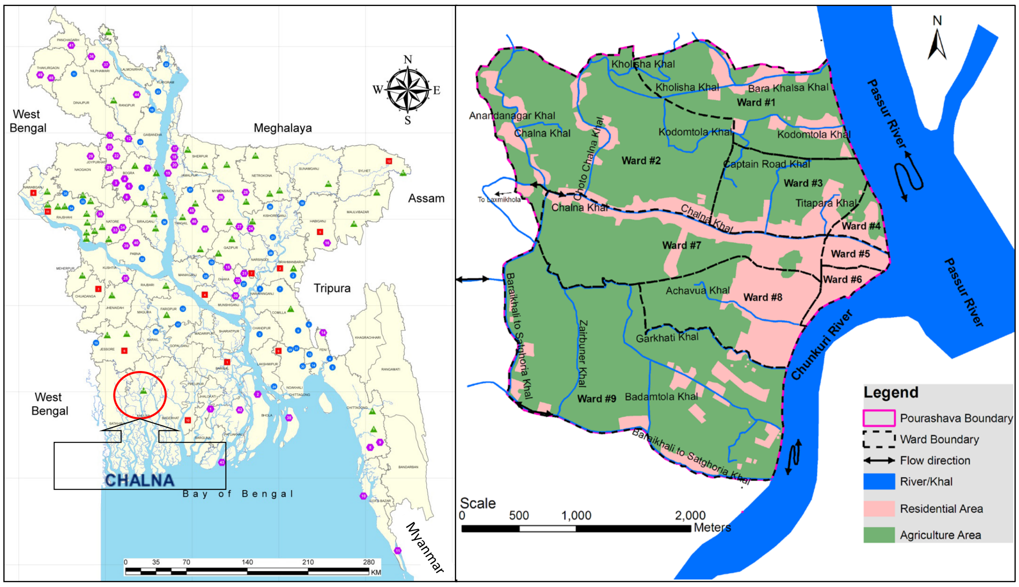

Figure 3: Location Land-use of Chalna Municipality

The change in elevation of most of the Municipality area is gradual. The land elevation of the Municipality effectively ranges between 0.64 mPWD and 3.19 mPWD. It is assessed that 11% land of the Municipality is below 1.08 mPWD while 47%, 62%, 72%, 83%, 90%, 99%and 100% of the land are below 1.39 mPWD, 1.54 mPWD, 1.69 mPWD, 1.99 mPWD, 2.44 mPWD, 3.04 mPWD and 3.19 mPWD respectively. Figure 4 shows area-elevation of the Municipality. The use of present Municipality's area can be broadly divided into lands for agricultural (76%) and non-agricultural (24%). Major settlements are in the areas of Ward Nos. 4, 5, 6 & 8 with some scattered settlements in Ward Nos. 1, 2, 3, 7 & 9.

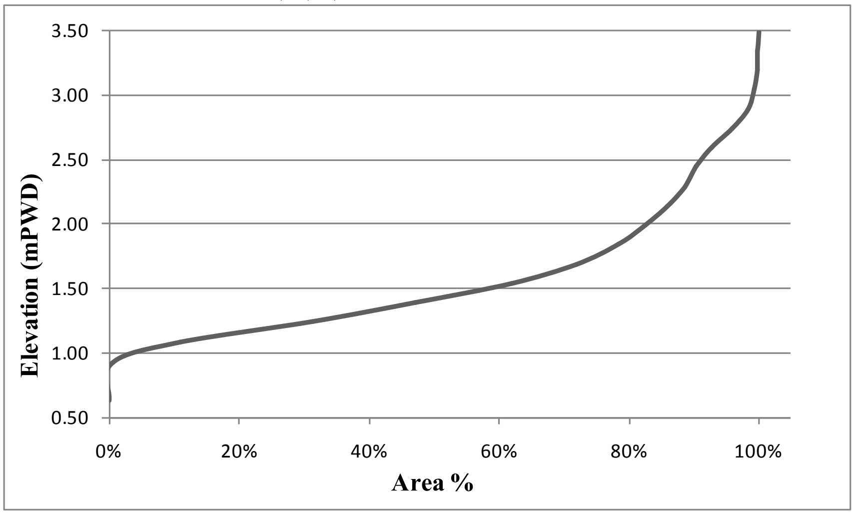

Figure 4: Area-Elevation curve of Chalna Municipality

3.2 River and Khal System

The Municipality stands on the right bank of Passur and Chunkuri rivers. The river Jhapjhapia runs on the north to west off the Municipality. The Passur and Chunkuri rivers run by the east of Municipality in south direction. These rivers surrounding the Municipality are tidal in nature. Regional river system adjacent to the Municipality is shown in Figure 5.

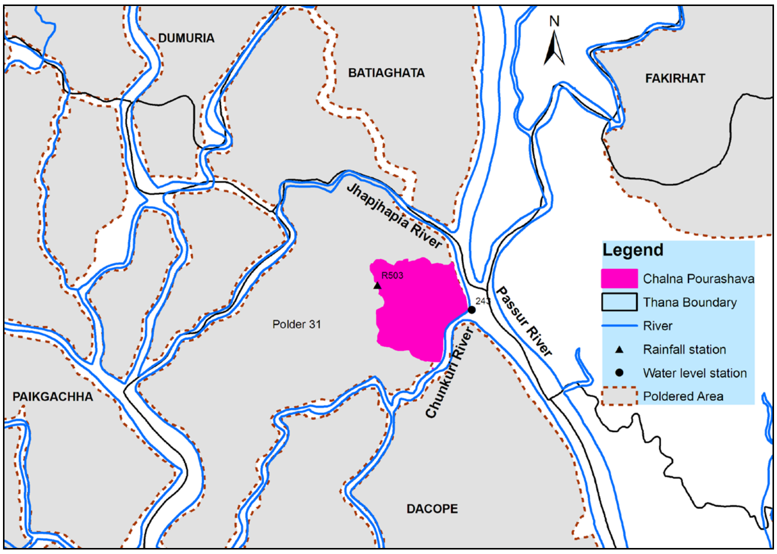

Figure 5: Regional river system surrounding the Municipality

The khal system of the Municipality is virtually non-tidal being subjected to the overall management of the Polder 31. Titapara Khal, Baraikali to Satghoria Khal, Chalna Khal, Choto-Chalna Khal, Kholisha-gate Khal, Kadom-Tola Khal, Achavua Khal, Garkhati Khal, Katakhali Khal, Barow Khal, Boro Kalsha Khal, Garkhati Khal, Zairbuner Khal, Annandanagor Khal and Captain Road side khal are the natural khals of the Municipality. The river and Khal system of Municipality is shown in Figure 6.

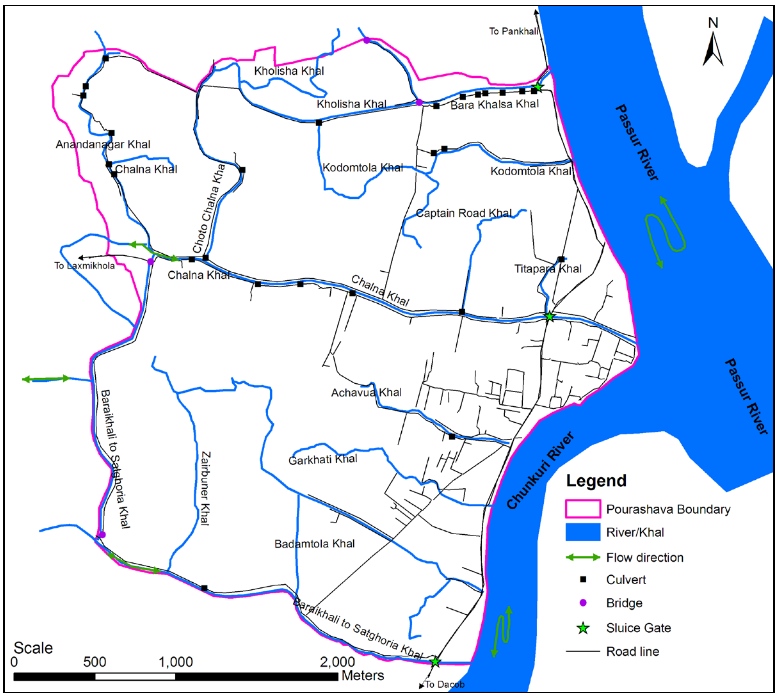

Figure 6: River and Khal system of the Municipality

Figure 7(a)

Figure 7(b)

Figure 7(a): Chalna Khal and Figure 7(b): Chunkuri River from Chalna Pourashava

3.3 Existing Drainage Network

There exist few lined and unlined drains within the Municipality. These can drain some local areas of the Municipality. The capacity and outfalls of existing drainage system is not planned with well defined consideration of drainage areas/zones for the whole Municipality. The lengths of existing lined and unlined drains are about 1.32 km. In absence of planned and adequate drainage system, the Municipality in some places suffer from drainage congestion and water logging after heavy rainfall

3.4 Rainfall Pattern

Chalna (R503) is a rainfall gauging station with reasonable length of records and is located nearest to the Municipality. The maximum and minimum rainfalls are 264 mm and 90 mm respectively with the event of yearly 1-day maximum rainfalls. The average yearly rainfall is about 2671 mm. About 74% rainfall occurs during the period from June to September.

3.5 Flood Pattern

The Municipality lies in the tidal basin of Passur-Sibsa River system. The nearest water level gauging is available at Chalna (243) on Passur River which is fairly calibrated by the regional model developed in IWM. The average year flood level for the Municipality is estimated to 1.13 mPWD in consideration with field visit, local people's opinion and satellite based flood map analysis. The major parts of the Municipality lie in the Polder 31 which is subjected to internal rain fed flood and the flood level inside the Polder is assessed to same as that of external average flood level. Some settlements of core area are relatively high and lie outside the Polder, and above the high tides. It is assessed that 66% of the area of Municipality is above the average flood level while the rest of the land is subjected to shallow depth of flooding.

4. DRAINAGE IMPROVEMENT PLAN

4.1 Classification of Municipality into Zones

The area of the Municipality has been planned for improvement under gravity drainage system. The whole Municipality has been divided into 14 zones for drainage improvement plan shown in Figure 7.

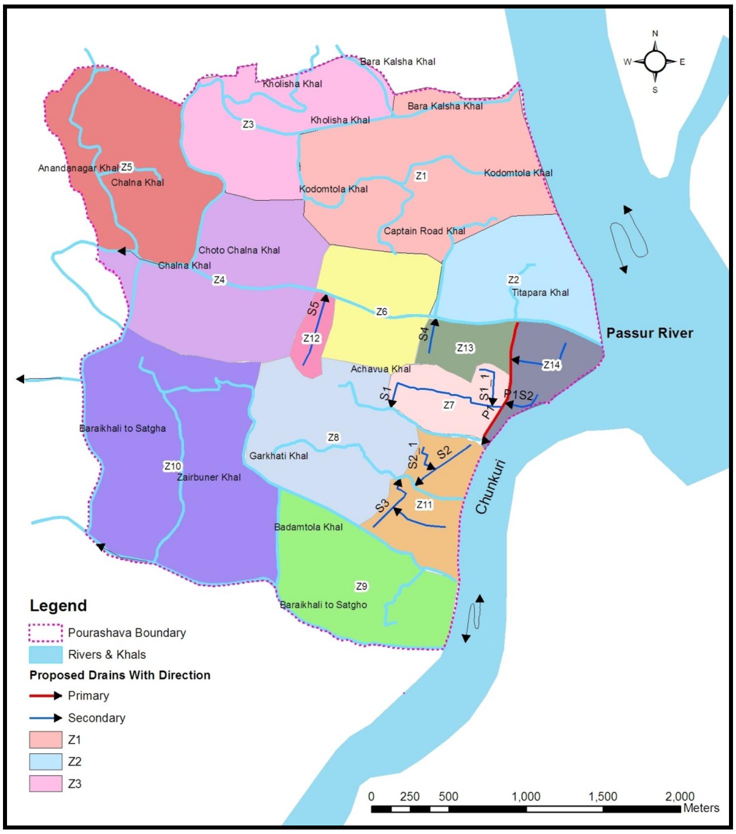

Figure 7: Drainage Zones of Chalna Municipality

The summary of runoff discharges of all 14 zones are given in Table 1. It is considered for the estimate of discharges that when storm drains are required in areas of the Municipality, such areas will have the characteristics of urbanization like mostly that of usual residential areas.

4.2 Proposed Drainage System

The drainage network of Chalna Municipality contains a few complex inter-connected networks of small drains. Most of the existing drains are found undefined and not routinely maintained. Major drains are considered based on identified outfalls, engineering survey data and existing drainage network map. Most of the drainage structures in the Chalna Municipality are culverts and bridges. Schematization of the model considers the primary channels, major drains and part of the network important for the system. Table 2 contains the list of drainage channels and drains included in the model.

Table 1: Design Discharge

Drainage Zone	Drainage Area (ha)	Discharge (m^3/s)
Zone-1	134	10.51
Zone-2	61	4.73
Zone-3	78	5.92
Zone-4	104	7.77
Zone-5	94	6.91
Zone-6	50	3.66
Zone-7	29	2.03
Zone-8	86	6.07
Zone-9	77	5.35
Zone-10	163	11.17
Zone-11	41	2.73
Zone-12	10	0.66
Zone-13	19	1.23
Zone-14	27	1.77

Table 2: List of proposed drainage system in Chalna Municipality

Sl. No	ID of Drain and Catchment	Length (m)	Area (ha)	Outfall
1	P1	873	6.92	To Chunkuri River
2	P1S1	452	7.04	To P1 Primary Drain
3	P1S2	256	1.88	To P1 Primary Drain
4	S1	89	16.94	To Achavua Khal
5	S1_1	321	4.37	To S1 Secondary Drain
6	S2	451	4.85	To Garkhati Khal
7	S2_1	227	1.86	To S2 Secondary Drain
8	S3	449	3.75	To Garkhati Khal
9	S3_1	372	4.41	To S3 Secondary Drain
10	S4	240	3.82	To Chalna Khal
11	S5	495	8.77	To Chalna Khal

The delineated catchments and drainage routes using GIS are shown in Figure 8. Topographic data such as land and road crest level and drain cross sections, hydrometric data such as water level and discharge, meteorological data such as rainfall and evaporation were collected to develop Chalna Drainage Model.

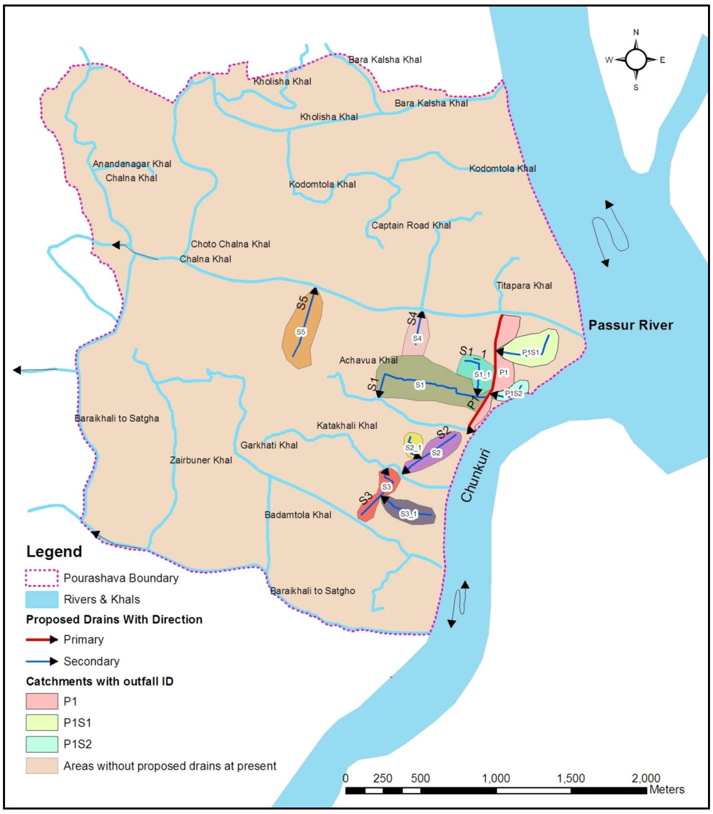

Figure 8: Drainage Routes and Catchments for Chalna Municipality

4.3 Rainfall-Runoff Model

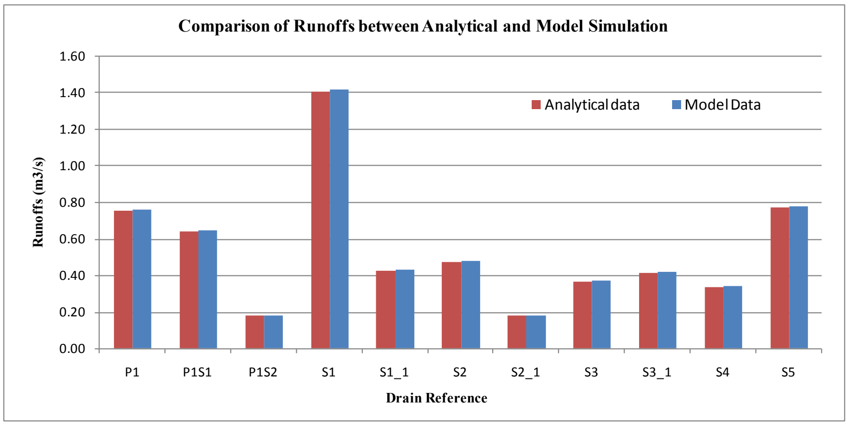

Figure 9: Runoff comparison between analytical and model simulation

Model study of hydrological analysis of the Chalna Municipality drainage system has been carried out using MIKE Urban model B concept of MIKE 11 hydrodynamic module. In this connection, GIS has been applied to delineate catchments and drainage routes of Chalna Municipality. A number of drainage areas have been delineated as catchments for the Chalna Municipality based on the area of interest. Existing roads, Digital Elevation Model (DEM), infrastructure, homestead, contour maps, natural canals and rivers in and around the Chalna Municipality and the outfalls have been considered in delineating the drainage routes and catchments. Figure 9 shows comparision of runoff calculated by analytical and modelling approach.

The model simulations have generated runoff in the drains draining towards the outfall channels proposed for the Chalna Municipality area. An analysis of 20 (1986-09) years runoffs from historical simulation of model is given in Table 3.

Table 3: Analysis of historical simulation of runoff

ID of Drain and Catchment	Average Flow (m^3/s)	Minimum Flow (m^3/s)	Maximum Flow (m^3/s)
P1	**0.77**	0.15	0.88
P1S1	0.65	0.14	0.77
P1S2	0.18	0.10	0.21
S1	1.42	0.21	1.35
S1_1	0.43	0.12	0.44
S2	0.48	0.10	0.50
S2_1	0.18	0.13	0.25
S3	0.41	0.14	0.36
S3_1	0.42	0.11	0.43
S4	0.34	0.11	0.33
S5	0.78	0.16	0.79

4.4 Proposed Design Parameters

Manning's Equation is used for the calculation of flow velocity and determining drain section. Design sections for all proposed drains are given in Table 4. A longitudinal section of a proposed drain is shown in Figure 10.

Table 4: Design Parameters

Drain ID	Chainage	Model Flows Q_d (m^3/s)	Bottom Width (m)	Actual drain depth* (m)	Design Capacity, Q_c (m^3/s)	Ground Level		Bottom level		Remarks
						U/S	D/S	U/S	D/S	
P1	0-242	0.21	0.65	0.85	0.30	2.32	2.15	1.30	1.10	*Pri., Rec., RCC
	242-530	0.46	0.80	1.00	0.49	2.15	2.25	1.10	1.05	Pri., Rec., RCC
	530-876	0.76	1.00	1.20	0.78	2.25	2.20	1.05	1.00	Pri., Rec., RCC
P1S1	0-452	0.65	0.80	1.15	0.98	2.23	1.76	0.85	0.60	Sec., Rec., RCC
P1S2	0-256	0.18	0.50	0.75	0.30	1.89	1.85	1.20	1.05	Sec., Rec., RCC
S1	0-60	0.11	0.70	1.15	0.25	2.49	2.15	1.62	1.00	Sec., Rec., RCC
	60-893	1.42	1.00	0.85	0.64	2.15	2.88	1.00	0.90	Sec., Rec., RCC
S1_1	0-321	0.43	0.65	0.95	0.44	2.02	2.10	1.25	1.10	Sec., Rec., RCC
S2	0-277	0.29	0.60	0.95	0.39	1.94	1.72	1.15	0.80	Sec., Rec., RCC
	277-451	0.48	0.70	1.00	0.53	1.72	1.77	0.80	0.70	Sec., Rec., RCC
S2_1	0-227	0.18	0.55	0.80	0.27	1.55	1.72	0.75	0.65	Sec., Rec., RCC
S3	0-183	0.19	0.60	0.90	0.36	1.71	1.84	0.90	0.80	Sec., Rec., RCC
	183-449	0.37	0.70	0.95	0.49	1.84	1.65	0.80	0.70	Sec., Rec., RCC
S3_1	0-373	0.42	0.70	0.95	0.49	2.21	1.82	1.25	0.80	Sec., Rec., RCC
S4	0-240	0.34	0.60	0.95	0.39	1.55	1.60	0.80	0.85	Sec., Rec., RCC
S5	0-495	0.78	0.80	1.20	0.83	1.64	1.95	0.85	0.65	Sec., Rec., RCC

Considering Freeboard. For Primary Drain 0.2m & Secondary Drain 0.15m
Pri=Primary, Rec=Rectangular, Sec= Secondary, RCC= Reinforced Cement Concrete

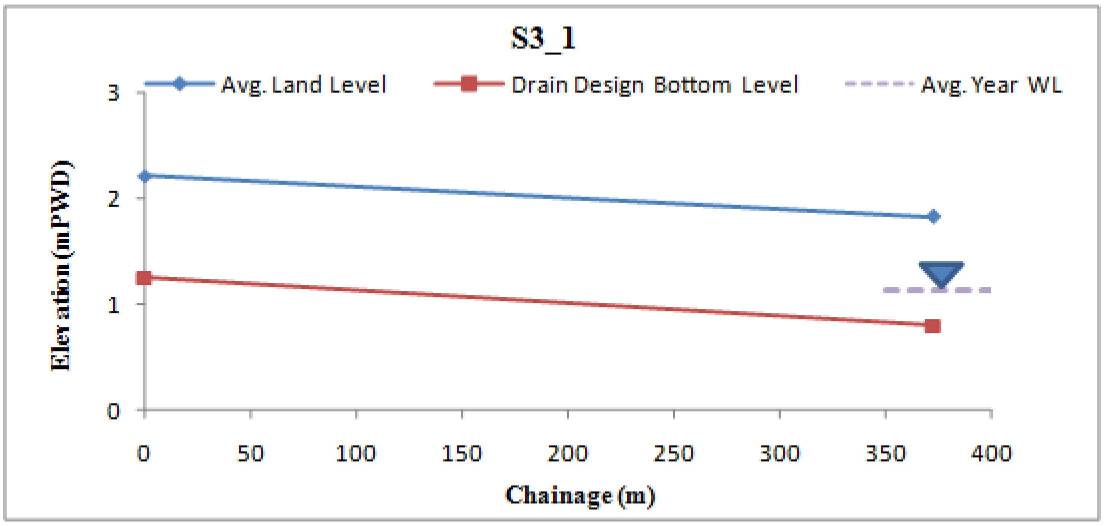

Figure 10: Sample Longitudinal Drain Section of drain S3_1

5. CONCLUSIONS

Analytical study of drainage system followed by the assessment of its performance in aid of model application is found to be a suitable and convenient method adopted for of the Municipality. S1, S1_1, S2, S2_1, S3, S3_1, S4, S5, P1, P1S1 and P1S2 are the major drains which are proposed for the storm drainage of the Chalna Municipality. P1, P1S1, P1S2, S1, S2, S3, S4 and S5 drainage systems have priority needs while S1_1, S2_1 and S3_1 drainage systems are proposed in view of near future needs for the Municipality. Raising of low land with earth fill above the flood level is a pre-requisite for the land to be brought under gravity drainage. It is recommended that such land is raised to the similar level of high land (not less than 2.5 mPWD) of the Municipality. Lack of social awareness is a huge concern for smooth functioning of the drains. Dumping of solid wastes should be prevented to keep the drain in flowing condition. Trash racks and silt traps shall have to be provided at regular interval or selected locations during construction of drains. The existing drains and also those will be constructed shall have to cleaned at regular interval. Maintenance and cleaning of drains shall have to be done at least once in every year specially before starting of monsoon season. The Municipality authority should monitor the water level at outfalls, record the drainage congestion area of each significant storm and maintain the existing river and khal so that natural channels/ drains are not encroached anyhow. The proposed drainage system is found adequate for storm drainage of the Municipality, and is recommended for detailed study and necessary modification in consideration of social, environmental, technical, economical and institutional constraints.

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REFERENCES

Ball J. and Babister M., (2010), Urban drainage modelling and the future of the urban rational method.

Berggren K., Svensson G. and Viklander M., (2008), Urban drainage and climate change: a problematic approach.

DPHE (Department of Public Health Engineering), (2010), Ground Water Management and Feasibility Study for 148 Municipalitys Having no Piped Water Supply; Component I- Mathematical Modelling for Safe Drinking Water Source Identification; Water Supply System Development; Dhamrai Municipality, Dhaka District, Bangladesh.

Haque, M.M. and Kamruzzaman, M. (2012), 'Enhancement of Drainage Situation in Small Township of Bangladesh using GIS: Akhaura Municipality as an Illustration', 1st International Conference on Advances in Civil Engineering-2012, CUET, Chittagong, Bangladesh.

Khan D.M., Shams K. and Hossain Md.K., (2012), Climate Change Impact on Urban Drainage in Coastal Belt of Bangladesh, 9th International Conference on Urban Drainage Modelling.

LGED (Local Government Engineering Department) (1998), Urban Drainage Manual, prepared by DHV and AQUA Consultants.

Loughlin G.O. and Stack B., (2012), Drains User Manual. A manual on the DRAINS program for urban stormwater drainage system design and analysis.

Rauch W., Krajewski J.L.B., Krebs P., Mark O., Schilling W., Schütze M. and Vanrolleghem P.A., (2010), Mathematical Modelling Of Integrated Urban Drainage Systems. Journal of Wat.Sci.Tech, Vol. 45, pp. 81-94.

Schmitt T.G., Schilling W., Saegrov S. and Nieschulz K.P., (2002), Flood risk management for urban drainage systems by simulation and optimization, 9th International Conference on Urban Drainage, Portland, region, USA.

Schmitt T.G., Thomas M. and Ettrich N., (2004), Analysis and Modelling of flooding urban Drainage systems, Journal of Hydrology (299)300-311.

Willems P., Nielsen K.A., Olsson J. and Nguyen V.T.V., (2010), Climate change impact assessment on urban rainfall extremes and urban drainage: Methods and shortcomings.

APPLICATION OF CCHE2D MATHEMATICAL MODEL IN THE GORAI OFFTAKE FOR TWO-DIMENSIONAL SIMULATION

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ABSTRACT

One of the major sources of fresh water flow in south west region of Bangladesh comes from the river Gorai which takes off from its parent river, the Ganges. Due to implementation of the Farakka Barrage in 1975, the dry season flows in Ganges River started to decline subsequently. As a result, flow through the Gorai started reducing and offtake got deposited hindering the safe passage of flow from the Ganges to the Gorai River. Due to this drying out of the Gorai offtake in dry period, intensity and spatial extent of salinity intrusion to the south west region has increased, which affects the groundwater and irrigation in the area. Therefore, it is always a vital issue to keep the river active and live, which can be done by different manmade activities. Again, due to manmade interventions the river flow becomes interrupted and thereby may cause the change in river morphology. In order to study the effect of any intervention in the river, application of 2D (Two dimensional) model in the Gorai Offake is inevitable. This study focuses mainly on the application of 2D model to assess different hydrodynamic characteristics of the river system. The model has been set with the recent bathymetry data collected from BWDB. The river system along the Ganges from 14.5 km upstream to 5 km downstream of the Gorai offtake with 5 km reach along the Gorai from the offtake has been selected for model set up. A 1D model (from the Hardinge Bridge to Sengram along the Ganges and from offtake to Janipur along the Gorai) has been developed in HECRAS to generate the boundary conditions for the 2D model. Calibration and validation has been carried out with the monsoon water level data of 2004. Different scenarios like effect of setting groyne, bank encroachment, dredging of the river bed are considered to be assessed using the present model. The morphological assessment like variation of sediment transport and bed level changes has also been studied. It is hoped that the results of the 2D model simulation will be helpful to suggest the effect of possible future development work to be implemented on this river system.

Keywords*: Gorai Offtake, 2D Modeling, Hydrodynamic simulation, Sediment transport, bed level changes*

1. INTRODUCTION

The Ganges is one of the largest river systems in the world. It rises south of the main Himalayan divide near Gangotri (elevation 4500 m) in Uttar Pradesh, India. On its way towards the sea, numerous tributaries join the Ganges River from India and Nepal. The river divides into two channels below Farakka. The right arm continues to flow south in West Bengal as the Bhagirathi-Hooghly on which Calcutta Port is situated. The left main arm enters Bangladesh 18 km below Farakka and joins the Brahmaputra River at Goalundo. In Bangladesh, the Gorai River is the main distributary which leaves the Ganges River about 65 km above the confluence of the Ganges and Brahmaputra Rivers.

The flow of the Gorai River is interrupted for more than a decade during lean flow seasons, causing harm to the environmental and socio-economic conditions along the river. Due to implementation of Farakka Barrage in the year 1975, the dry season flows in Ganges River started to decline significantly. As a result of the reduction of flow in the Ganges, the conveyance of the flow through the Gorai started reducing and offtake got deposited. Continuous siltation at the offtake area ultimately closes the flow to the Gorai totally for five months of the dry period.

In order to restore the Gorai River offtake and ensure freshwater flow towards the south west region, BWDB has taken up steps to carry out dredging from the offtake for a length of 30 km of the river. To understand the dynamics of flow of water and sediment, especially at the Ganges-Gorai offtake, and to devise suitable other options for this critical condition, mathematical modeling technique can be effectively applied. Mathematical modeling can be done to establish suitable strategy for capital dredging operation as well as assess backfilling rate and required dredging volume for maintenance dredging, and to test alternative interventions which may be required to keep the Gorai flow uninterrupted throughout the year.

Figure 1: View of study area from 'Google Earth'

The objectives of this study are to develop a 1D model of the river network in HECRAS and to check its capability of sediment analysis, to develop a model of the Gorai offtake in CCHE2D, to assess the hydrological and morphological characteristics of the Ganges-Gorai offtake and to observe the morphological changes of the offtake after one monsoon flow.

2. MATERIALS AND METHODS

The study comprises a one dimension model using HEC-RAS and a two dimension model using CCHE2D. Here the methodology is illustrated about both the model development and result analysis.

2.1 One Dimension Model Development

For this study one dimensional model is developed using HEC-RAS (Hydrologic Engineering Centers River Analysis System). The model geometry is developed firstly, and then the initial and boundary condition is applied for the year 2004. The model is also calibrated and validated using observed data. From this model result, boundary condition for the Two Dimension model is extracted.

2.1.1 Model Geometry

The HEC-RAS software model is developed with a 72.6 kilometer long Ganges River from upstream at Hardinge Bridge and the downstream is located at Sengram. Gorai River of 30 kilometer from Gorai Offtake to Janipur is taken into consideration. Hec-GeoRAS is used to link the HEC-RAS model with ArcGIS 10.0 and for XS Location. The model study area and network geometry is shown in figure 2.1.

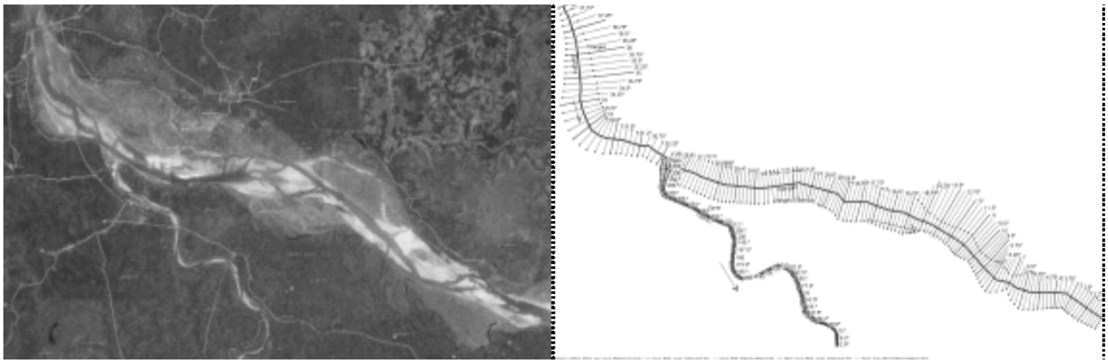

Figure 2: Study area & Model Setup network

2.1.2 Initial and Boundary Condition

For unsteady flow simulation HEC-RAS requires Boundary conditions at all open boundaries and Initial conditions at the junctions are applied. Here the model has three open boundaries. Upstream boundary at Hardinge Bridge is given as discharge data of the year 2004; downstream boundary condition of Ganges River is provided Water Level at Sengram of same year. Downstream boundary of Gorai River is given as Water Level of Janipur of same year. All the data are measured data of Bangladesh Water Development Board. For initial condition at junction Gorai-offtake , Boundary conditions were taken into account, the initial values of each corresponding boundary value is taken. Discharge Hydrograph at Hardinge Bridge, Water Level Hydrograph at Sengram and Water Level Hydrograph at Janipur are shown in figure 2.2 accordingly.

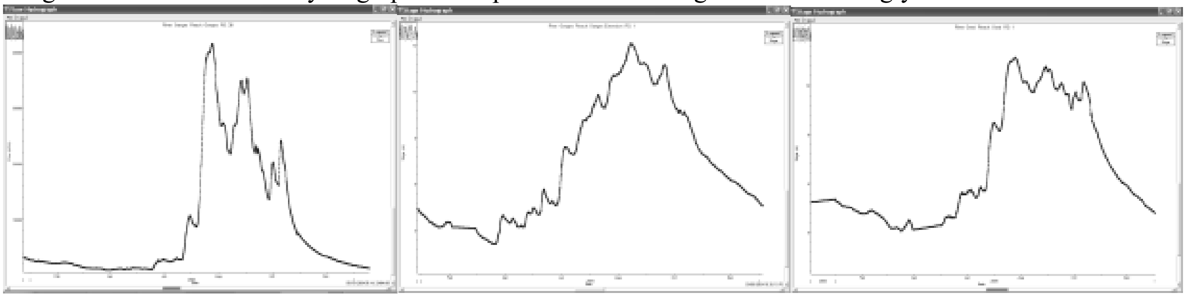

Figure 3: Upstream & Downstream Boundary Hydrographs

2.1.2 Calibration and Validation of Model

Before the application of any model, it is necessary to calibrate it with the observed data. Model Calibration is a process of comparing the model to actual system behaviour until model accuracy is judged to be acceptable. In our study the HEC-RAS Model is calibrated with the measured Water Level of Gorai Railway Bridge from January 2004 to June 2004. Similarly for model validation the Water Level data from June 2004 to December 2004 is at Gorai Railway Bridge is applied.

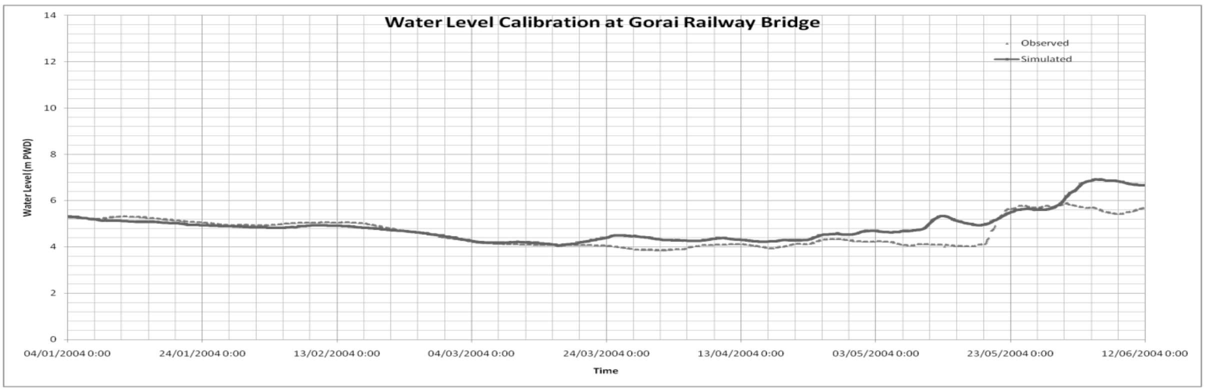

Figure 4: Calibration of HEC-RAS Model for January 2004-June 2004

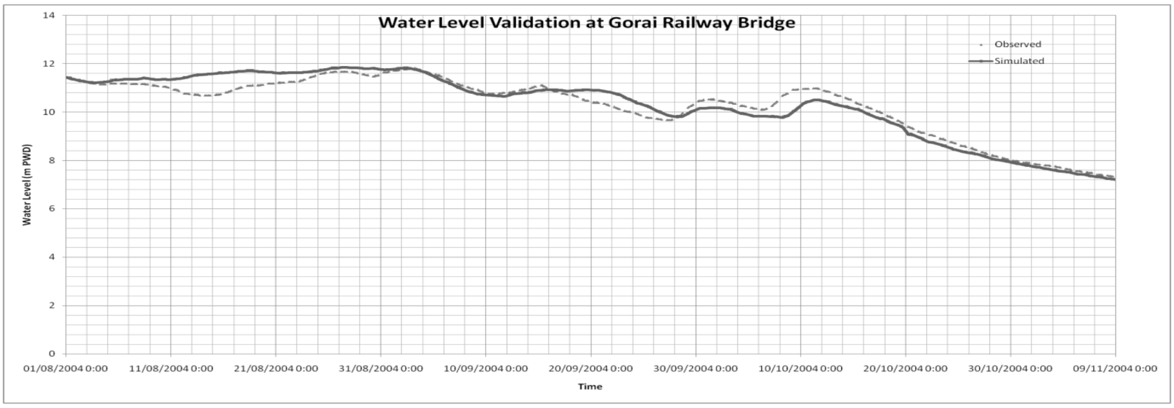

Figure 5: Validation of HEC-RAS Model for June 2004-October 2004

2.1.3 Sediment Analysis

Sediment properties had been given to run the sediment analysis. But having two reaches leaving the junction, the present river network could not be analyzed by HECRAS. It cannot handle sediment budgeting for such river network. More easily the sediment flow splits is not possible in HEC-RAS. Figure 2.5 shows the error message of HEC-RAS window for sediment analysis.

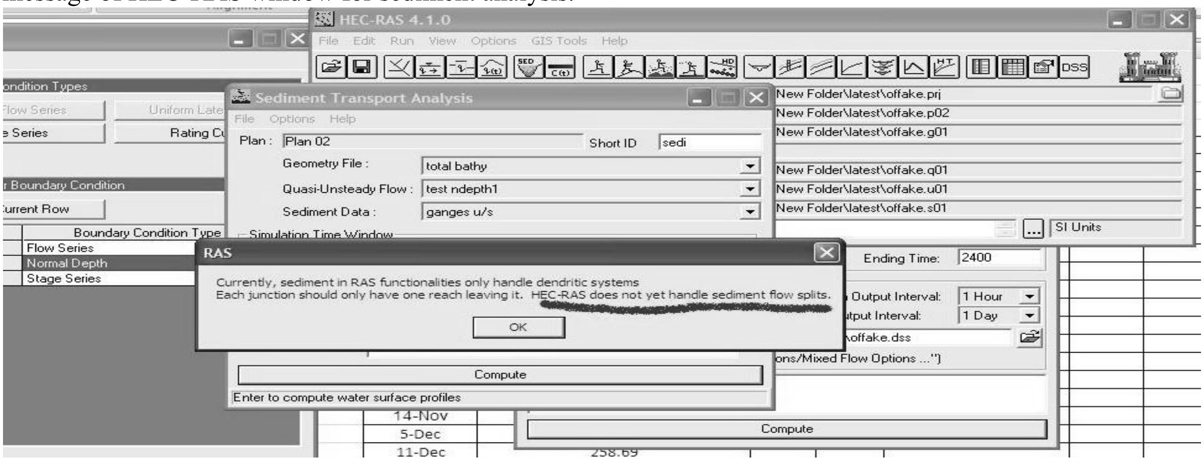

Figure 6: Error message in sediment analysis

For this reason a Two Dimension Model is developed and the boundary data of upstream and downstream is applied to the model extracted from result time series discharge and water level data of One Dimensional Model.

2.2 Two Dimension Model development

A brief is given about background of CCHE2D model and methodology to develop the model. Mesh generation, Sediment parameters and boundary conditions is discussed below.

2.2.1 Equations for 2D Modeling

Equations which are used in CCHE2D software are the equations for flow and sediment in open channels and rivers as the followings. Equations for wave dynamic model in an open channel are Saint Venant equations. Equation (1) is the Continuity equation and Equation (2) is the Momentum equation.

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q \qquad (1)$$

$$\frac{\partial}{\partial t}\left(\frac{Q}{A}\right) + \frac{\partial}{\partial x}\left(\frac{\beta Q^2}{2A^2}\right) + g\frac{\partial h}{\partial x} + g(S_f - S_0) = 0 \qquad (2)$$

In these equations, x and t represents time and place axles. A is flow space, Q is flow discharge, h is flow depth, S_0 is steep of the river bed, b is correction factor of momentum factor, g is gravity speed, q is discharge to width unit and S_f represents frictional steep. In the dynamic wave method, we use the complete momentum equation. Complete momentum equation together with the continuity equation can be only solved by numeric methods.

The momentum equation for wave spreading model is Equation 3.The equation for non uniform sediment transport is Equation 4.

$$\frac{\partial h}{\partial x} + S_f - S_0 \tag{3}$$

$$\frac{\partial (AC_{tk})}{\partial t} + \frac{\partial Q_{tk}}{\partial x} + \frac{1}{L_s}(Q_{tk} - Q_{t*k}) = q_{lk} \tag{4}$$

Where C_{tk} is the mean (average) of sediment density for the size of k units, Q_{tk} is the rate of actual carried alluvia for the size of k units, Q_{t*k} is the capacity for carrying sediments, L_s the length of the distance that sediment is carried inconstantly and q_{lk} the side discharge or output sediments in width unit.

In order to estimate the sediment, this software uses four methods as follows: Modified Equation of Eakers, and White, Modified Equation of Angelond and Henson, Equation of Woo et al., Equation of Yan.

2.2.2 CCHE2D Model

CCHE2D is an aggregated software package created in 2005 by Wang, Jia and San in the National Centre of Calculation and Hydraulic Engineering (NCCHE) under the supervision of University of Mississippi, USA. This model is a two dimensional hydraulic model that is created for analyzing and simulation of flow hydraulic, sediment transport and morphology processes in open flows. The model uses average equations of Reynolds for solving flow area in depth. Two zero models of parabolic distributed equations and the prantels mix length model and also the model of two equations k-ε are used for modeling of distributed flow.

The CCHE2D has two important categories: CCHE2D Mesh generator and CCHE2D GUI (Graphical User Interface). The Mesh generator allows the user to introduce the geometric condition and structures of environment to the model and then proceeds to create the structure's network. Then, using the GUI model, user can observe hydraulic parameters of flow, sediment, boundary condition, parameters needed for simulation and also the output results.

2.2.3 Mesh Generation and bathymetry

This research is made in the area of Ganges-Gorai River System (Upstream at Hardinge Bridge and both downstream ends are at 5 km from offtake). Here the 2D model has been applied to calculate the flow velocity distribution, shear stress and sediment transport. These results are the pre-requisite for in depth analysis of different phenomena. A RL orthogonal mesh has been generated which contains 2 blocks in the model area. To connect the blocks special care was taken. It contains total 84 grids in I direction and 100 grids across the model area. Minimum cell length in I and J direction are 12.99 m and 47.53 m consecutively. This mesh was used to interpolate bathymetric data and assign values. Triangular interpolation was done to generate the bathymetry for the model.

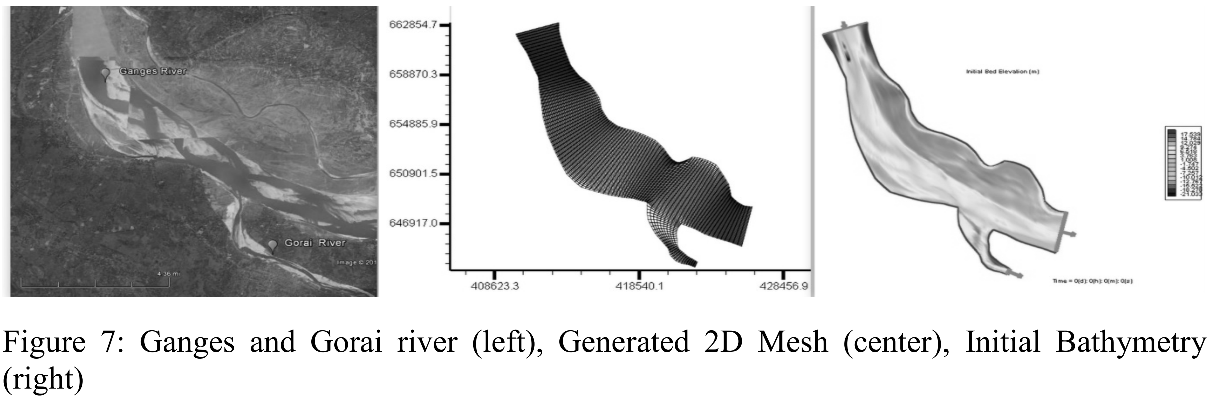

Figure 7: Ganges and Gorai river (left), Generated 2D Mesh (center), Initial Bathymetry (right)

2.2.4 Boundary conditions

The model was run for the Two months (14 July 2004 - 14 September 2004) of Monsoon days. Boundary condition were taken from the HECRAS output, the same as the previous model that is discharge boundary at the upstream direction and water level boundaries at the downstream ends.

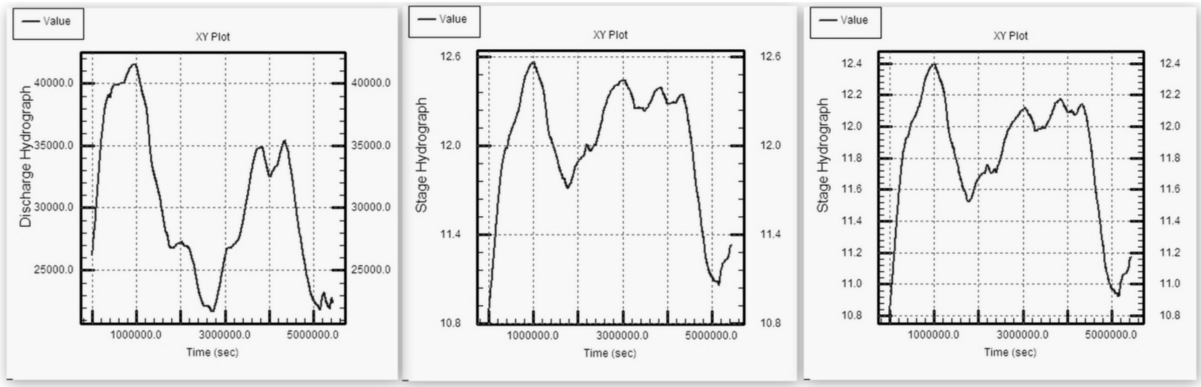

Figure 8: Boundary Conditions: Ganges u/s (left), Ganges d/s (center), Gorai d/s (right)

2.2.5 Sediment Parameters

After the hydrodynamic run, sediment transport simulation had been done from that flow field. Particle curves of suspended sediment, bed sediment, suspended sediment discharge and bed load discharge were entered to software. Initial conditions and sediment boundary conditions were determined for the software and the sediment load was analyzed.

2.2.6 Executing the model

Considering the peak flood crossing time in the studied area, the simulation time is set equal to 5439000 seconds. The specified total time step is equal to 54390. After completing other adjustments and inserting necessary information, the software starts getting runs for monsoon of 2004.

2.2.7 Calibration and Validation

Simulated water level was exported and processed using GIS tool for calibration. The calculated and observed water level showed quite good agreement with slight variation at some locations for the monsoon. The model predicts the water levels quite well, sometimes it is little bit overestimated. Variable roughness values are used throughout the model which acted as the calibration parameter.

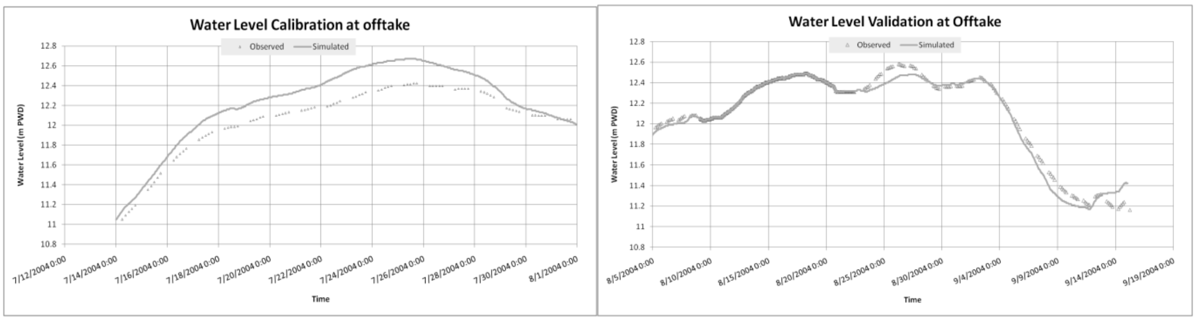

Figure 9: (a) Calibration for July and (b) Validation for August-September at Gorai Offtake

3. RESULT AND DISCUSSION

After introducing geometric and hydraulic specifications of flow to the model and running the model, one can observe the modeling results in different forms. In this research, the effects of parameters like velocity, shear tension, specific discharge on the study area and river offtake will be studied. The results of velocity distribution and their values for the study period are shown in Figure 3.1.

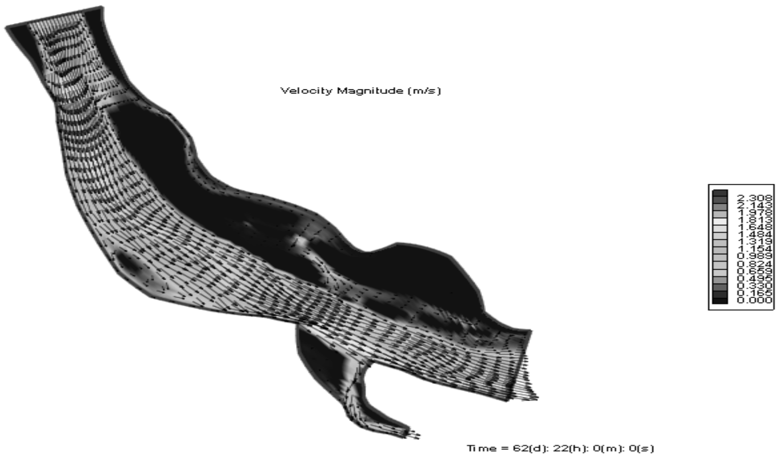

Figure 10: Velocity magnitude with velocity vectors after simulation period

Bed shear stress and specific discharge are other important hydrodynamic properties of river. The results are shown in the following figure.

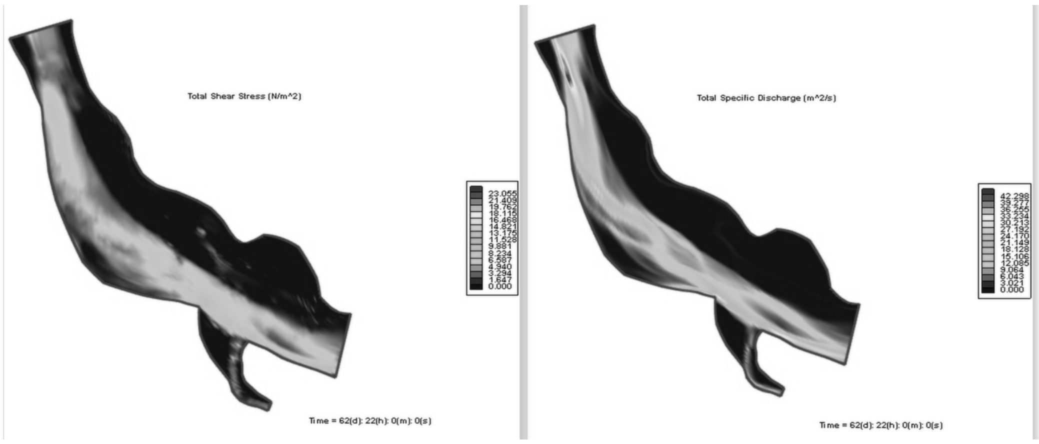

Figure 11: Total shear stress (left) and total specific discharge (right) after simulation period

The results of sediment analysis give the bedload transport rate and suspendedload concentration. These results are shown in the following figure.

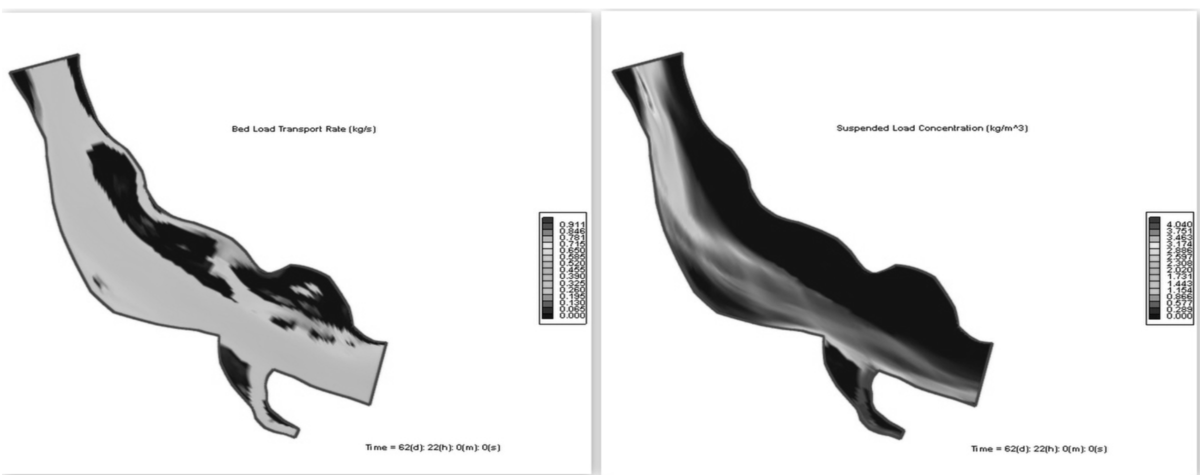

Figure 12: Bedload transport (left) and suspendedload cncentration (right) after simulation period

From morphological study the most important result is the bed change over the study area which is presented here in the figure below. One of the important locations of the study area is the Gorai offtake. Siltation at this site is observed in the model result which is depicted in figure 3.4.

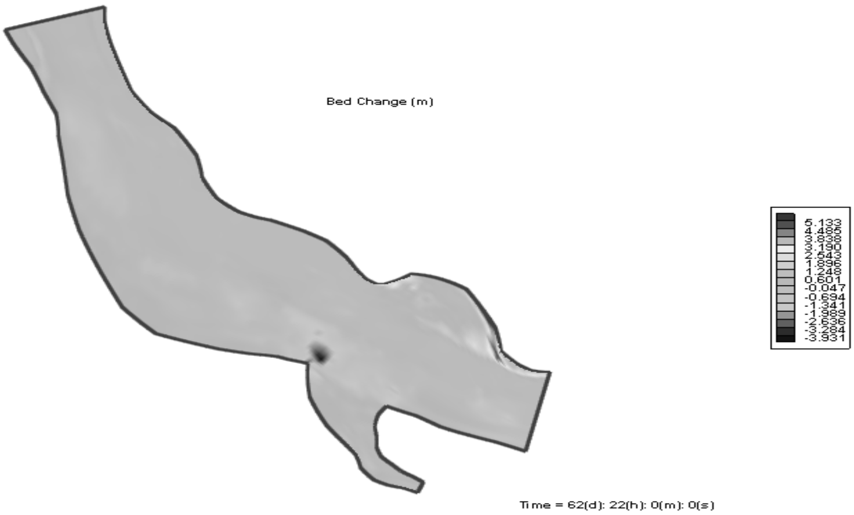

Figure 13: Bed change in the study area after simulation period

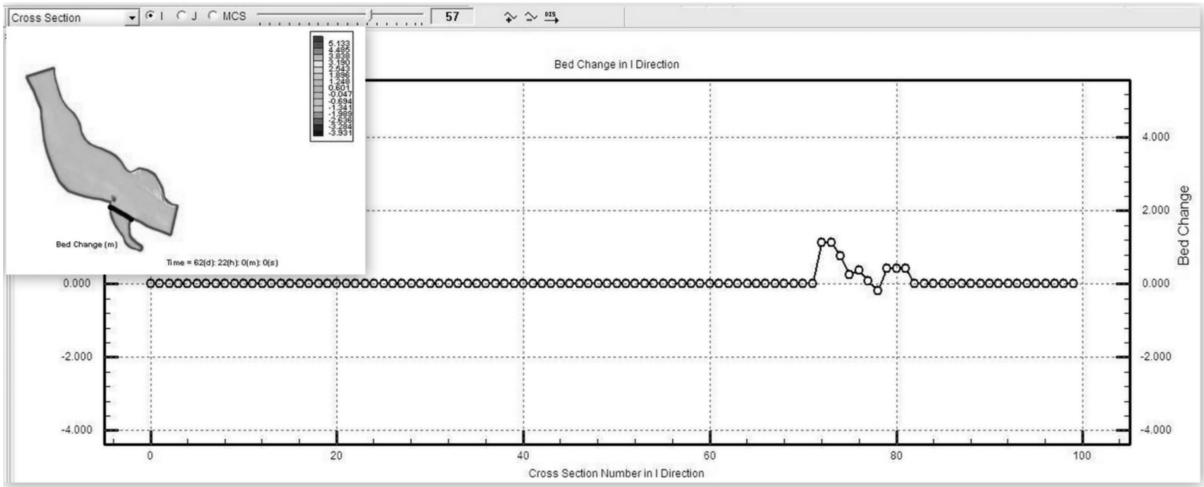

Fig 14: Siltation at Gorai offtake

The summery of the flow result and sediment result can be observed from following table.

Table 1: Summery of flow and sediment result

Flow Result	
Maximum Velocity magnitude (m/s)	2.31
Maximum Total specific discharge (m^2/s)	42.3
Maximum Total shear stress (N/m^2)	23.1
Sediment Result	
Maximum Bed Change (m)	5.13 and -3.93
Maximum Bedload Transport Rate (kg/m/s)	0.911
Maximum Suspendedload Concentration (kg/m^3)	4.04

4. CONCLUSION

This paper investigates the unsteady pattern of Ganges-Gorai River flow as well as variations of river bed elevation and sediment transport due to 2004 monsoon event using CCHE2D. Two-dimensional (2D) depth averaged model has been developed for which calibration and validation of the model shows significant compliance with the observed data. Model results are generated for velocity filed, shear stress and specific discharge with assessment of morphological changes such as erosion or sedimentation of the river. Maintenance dredging has been recommended for the resulting deposition at Gorai offtake. Through the application of this model, the change in river behavior under different scenarios can also be assessed for proper planning of any development project on the Ganges-Gorai River system.

ACKNOWLEDGEMENTS

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REFERENCES

Alam S. and Matin M. A.(2012), "Application of Delft3D Mathematical Model in the river Karnafuli For Two-Dimensional simulation", Proceedings of 1st International Conference on Advances in Civil Engineering, CUET, Bangladesh.

Hydrologic Engineering Center, US Army Corps (2010), "HEC-RAS River Analysis System", Hydraulic Reference Manual.

Institute of Water Modelling (2011),"Mathematical Modelling for Feasibility Study of Ganges Barrage Project", Draft Final Report.

Kamanbedast A. A. et al.(2013,), "Estimation of Sediment Transport in Rivers Using CCHE2D Model (Case Study: Karkheh River) ", Indian Journal of Science and Technology, Vol: 6, ISSN:0974-6846.

Mirza M. Q. (1997), Hydrological changes in the Ganges system in Bangladesh in the post-Farakka period, Hydrological Sciences Journal, Volume 42, Issue 5.

Zhang, Y. (2005), "CCHE2D-GUI-graphical user interface for the CCHE2D model." Technical Report No. NCCHE-TR-2005-03, University of Mississippi.

COMPUTATION OF DISCHARGE AND FLOW VOLUME FOR DIFFERENT FLOODING SCENARIO IN THE LOWER MEGHNA ESTUARY

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ABSTRACT

The Ganges-Brahmaputra-Meghna (GBM) delta is one of the most dynamic tide dominated delta in the world. The flow generated in the upper catchment of the GBM basins is passing through the estuarine systems of the GBM delta. The particular feature of these estuarine systems is characterized by extreme flow concentration in one single estuary, the Lower Meghna Estuary. Due to highly dynamic nature and a huge flow volume, it is difficult to measure the discharge in the Lower Meghna estuary. Till now, there is no measuring station and no reliable estimate of total flow volume which is passing through the Lower Meghna estuary. In this study, a 1D dynamic model (HEC RAS unsteady) is applied to compute the discharge in the Lower Meghna estuary. In the schematized model network, all the major rivers of the delta are included that contribute flow to the Lower Meghna estuary. The model is validated using measured water level available at the Lower Meghna estuary. The validated model is used to generate the temporal variation of discharge and the flow volume passing through the Lower Meghna estuary for different flooding scenario.

Keywords: *Ganges-Brahmaputra-Meghna, Estuarine systems, Flow volume, Lower Meghna, 1D model, Discharge*

1. INTRODUCTION

The Ganges–Brahmaputra–Meghna (GBM) river systems is the third largest freshwater outlet to the world's oceans; it is exceeded only by the Amazon and the Congo rivers (Coleman, 1967). The headwaters of both the Ganges River and the Brahmaputra River originate in the Himalayan mountain range in China. The Brahmaputra and Ganges encompass a number of countries in the South Asian region, including China, India, Nepal, and Bangladesh. The Brahmaputra, after travelling about 1800 km through Tibet and India, enters northern Bangladesh through the northern border. The Ganges flows for about 2000 km through India, and enters through the western side of Bangladesh. After flowing into Bangladesh, the Ganges, Brahmaputra and Upper Meghna rivers join and flow into the Bay of Bengal as the Lower Meghna Estuary. Together with the Tetulia and Lohalia estuaries, the Lower Meghna estuary forms the Eastern Estuarine System (EES) (Rahman et al, 2013). The other systems are the Central Estuarine System (CES) and the Western Estuarine System (WES) (Figure 1).

Although Lower Meghna Estuary is the main drainage route of the flow volume generated in the upstream catchment of the GBM basins, no measured discharge data is available in this estuary. Model studies are available indicating flow distribution among different channels inside Meghna estuary (MES, 2001). But seasonal variation of flow volume in different flooding conditions (average, wet and dry) in the Lower Meghna estuary is still not available. In this study a one dimensional numerical model (HEC RAS unsteady) has been applied to compute the discharge and flow volume in the Lower Meghna estuary. There were several attempts of applying models of different complexities in this region, i.e., 1D model (Chowdhury and Haque, 1988; Islam and Chowdhury, 2002), 2D model (MES, 2001, Rahman et al., 2013) and 3D model (Hussain, 2009). 1D models are useful for volumetric flow analysis, 2D models for horizontal flow and transport fields and 3D models for complex secondary current interactions. As this study focuses mainly on the volumetric flow analysis (both discharge and flow volume), a 1D model is used. The study aims at analyzing the temporal pattern of discharge variation and the volumetric flow distribution in the Lower Meghna estuary.

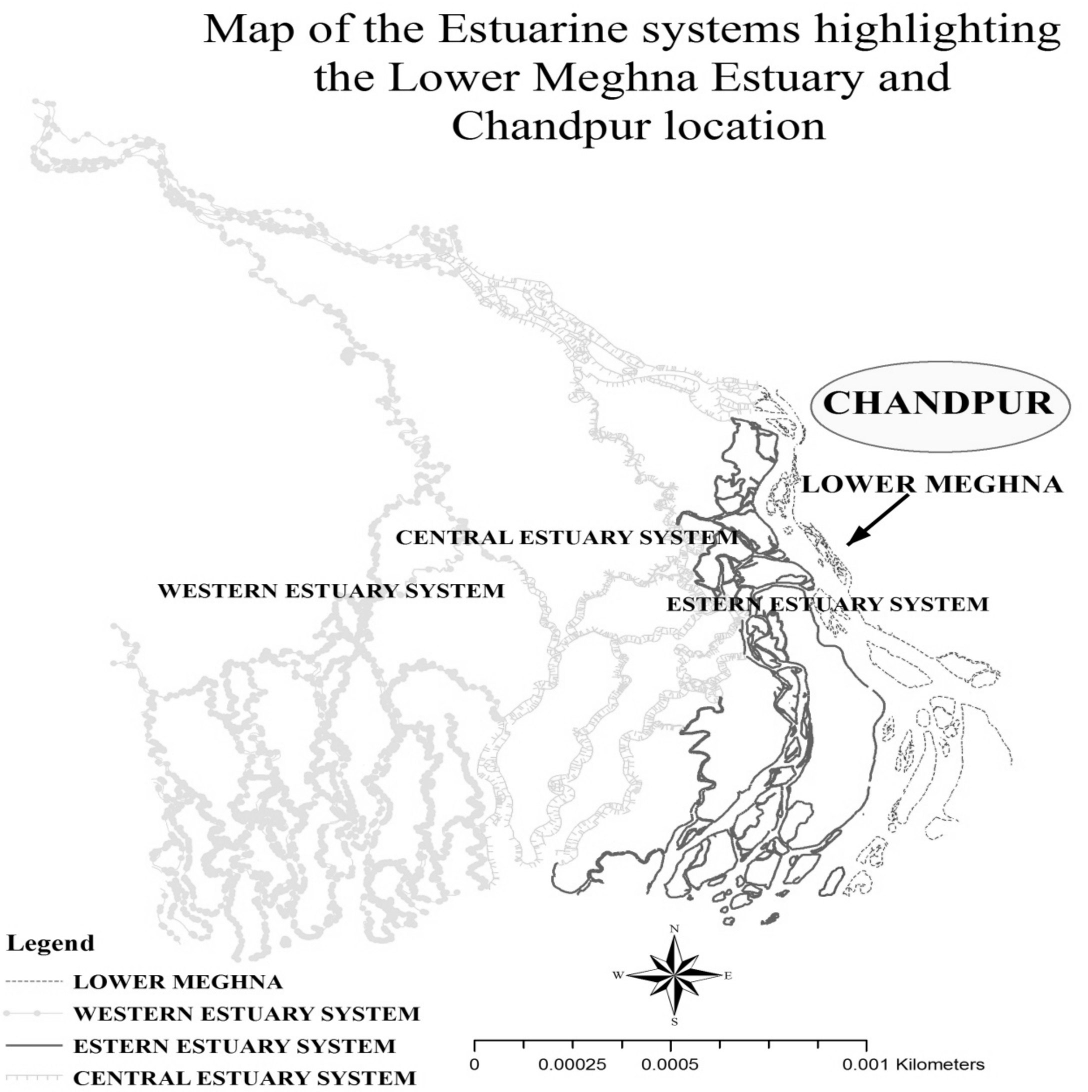

Figure 1: Map of the GBM estuarine systems highlighting the Lower Meghna Estuary and Chandpur location

2. THE HEC RAS UNSTEADY MODEL

Developed by the U.S. Army Corps of Engineers, HEC-RAS allows users to perform one-dimensional steady and unsteady flow calculations (HEC, 2010). In HEC-RAS steady state simulation, water surface profiles are computed from one cross-section to the next by solving the standard step iterative procedure to solve the energy equation. On the hand, the unsteady version of the HEC RAS solves the St. Venant's equations of mass and momentum conservation (HEC, 2010). The energy equation is intended to calculate water surface profiles for unsteady gradually varied flow. The numerical solution is performed by using implicit finite difference method. The model can accommodate branching of river networks with multiple junctions. The model requires upstream and downstream boundary conditions and detailed bathymetry of the river network. Time series of discharge has to be specified as the upstream boundary condition and time series of water level has to be specified as the downstream boundary condition.

3. SCHEMATIZED REACH OF THE MODEL DOMAIN

The estuarine network in the model domain is constructed based on the significance of the channels in the estuarine systems and availability of the cross section data. All the major rivers namely the Ganges, the Brahmaputra, the Old Brahmaputra, the Padma, the Upper Meghna and the Lower Meghna are included in the model network. The major distributaries in the estuarine systems, the Gorai River and the Arial Khan River are also included in the schematized network.

The three important spill channels from the Lower Meghna estuary that discharges into the CES along with the Tetulia estuary are in the network. The contribution of the three estuaries namely the Buriswar, Bishkhali and the Baleswar which are basically part of the CES are also included in the network. The schematized river and estuarine network in the model domain is shown in Figure 2.

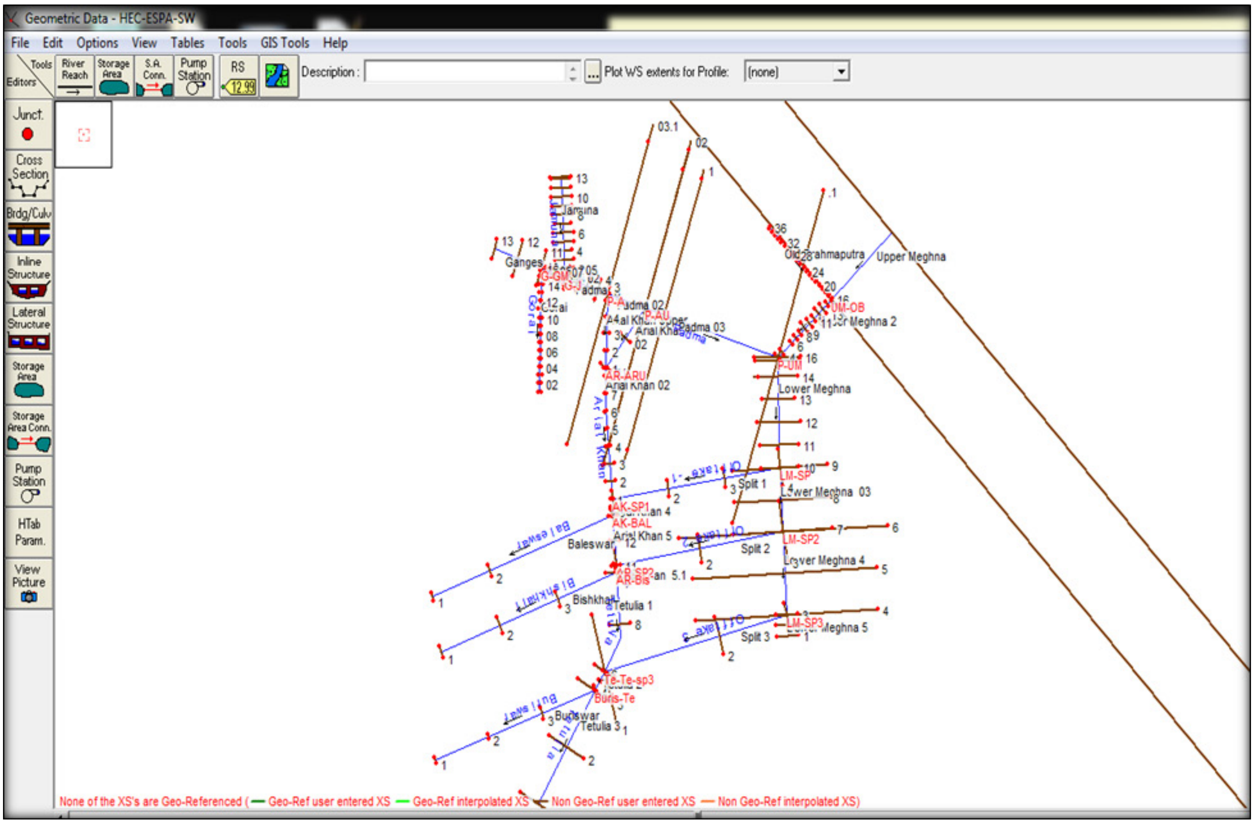

Figure 2: Schematized HEC-RAS Model Network for discharge computaion

4. 1D MODEL SETUP AND VALIDATION

HEC-RAS 4.1 version is used to compute the seasonal discharge and flow volume in three different flooding conditions in the Lower Meghna estuary. The schematize river and estuaruries as described above are included in the model setup.

The geometric data along with upstream discharge and downstream water level data have been collected from Bangladesh Water Development Board (BWDB) for different years. For three spilt channels of Meghna the cross section data is considered from Google earth elevation profile. In addition, few elevation profiles are drawn using cross section data from years having equivalent flooding conditions. More to this point, erroneous depths are adjusted manually to manage to flow to drain out. The distance between stations for discrete river reaches are measured from BWDB maps and Google earth using ArcMap 10. In most of the cases, the station to station distance is 6 km. The measured discharge values at the Ganges, the Brahmaputra and the Upper Meghna are used as the upstream boundaries. The measured water levels at the most seaward limits of the Lower Meghna estuary, the Tetulia estuary, the Burishwar estuary, the Bishkhali estuary, the Baleswar estuary and the Gorai River are used as the downstream boundaries of the model. The initial flow conditions are set using opening dishcage value in respective rivers.

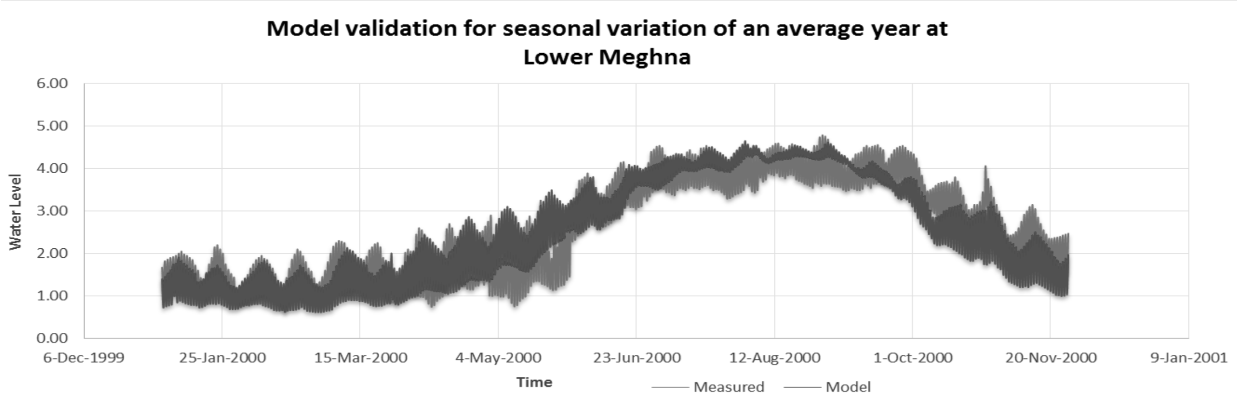

Figure 3: Model calibration for an average flood year at Chandpur of the Lower Meghna Estuary

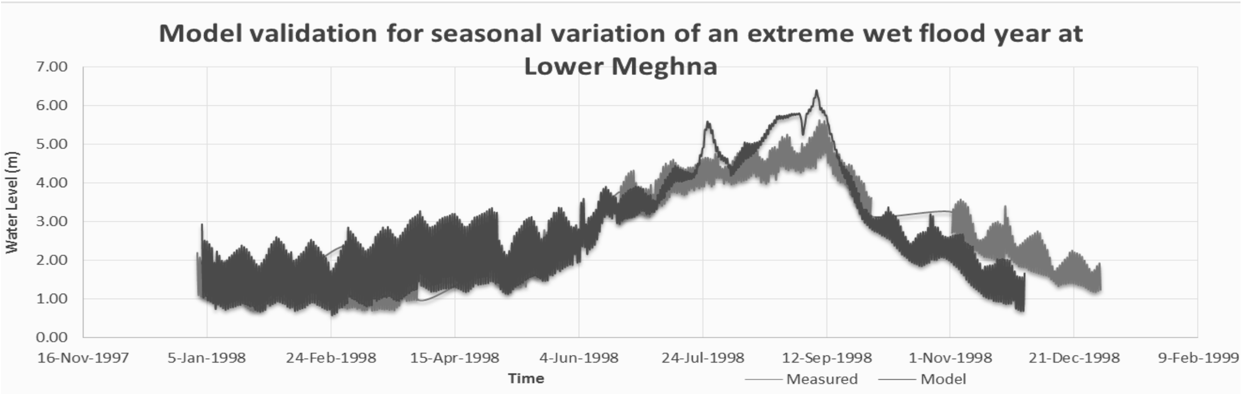

Figure 4: Model validation for an extreme wet year at Chandpur of the Lower Meghna Estuary

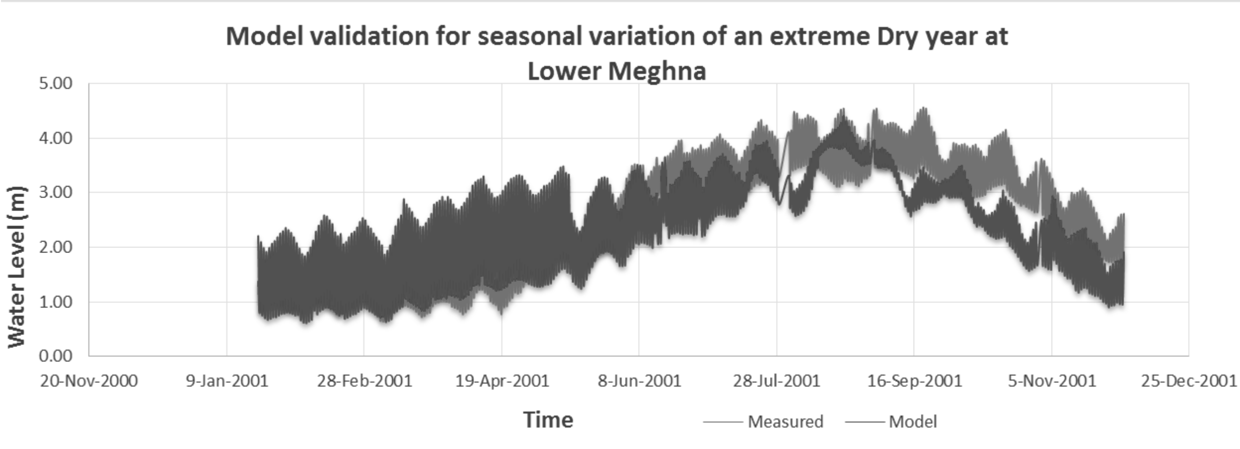

Figure 5: Model validation for an extreme dry year at Chandpur of the Lower Meghna Estuary

Chandpur (Figure 1), which is located just after the confluence of Padma River and the Upper Meghna River and upstream of the three Offtakes, is used as the water level station for the model calibration and validation. The model is calibrated for the average flooding scenario. The Mannings' roughness coeffcient n is used as the calibration parameter. After performing several calibration exercise, a roughnes co-efficient of 0.015 is found to be best representative roughness value for the Lower Meghna estuary. The model is finally validated with n=0.015 for average flood, extreme wet and extreme dry conditions and the results are shown in Figures 3,4 and 5. The visual observation shows a reasonable model performance for the three flooding scenario. Although the monsoon flows seem to be slightly under or over predicted in different flooding scenario.

5. DISCHARGE AND FLOW VOLUME IN THE LOWER MEGHNA ESTUARY

If discharges through the Gorai River, the Arial Khan River and some small channels from the Padma rivers are considered insignificant compared to the combined flow of the Ganges-Brahmaputra-Upper Meghna, the entire flood flow of all the major rivers of the delta can be considered to pass through the Lower Meghna estuary. Chandpur station (Figure 1) can be selected to compute this combined flow. Discharges are computed for an average flood year, for an extreme wet year and for an extreme dry year to represent the average, maximum and minimum flow situations. The year 2000 is selected as the average flood year, year 1998 is selected as the extreme wet year and the year 2001 is selected as the extreme dry year considering the percentage of the lands that those particular floods inundated.

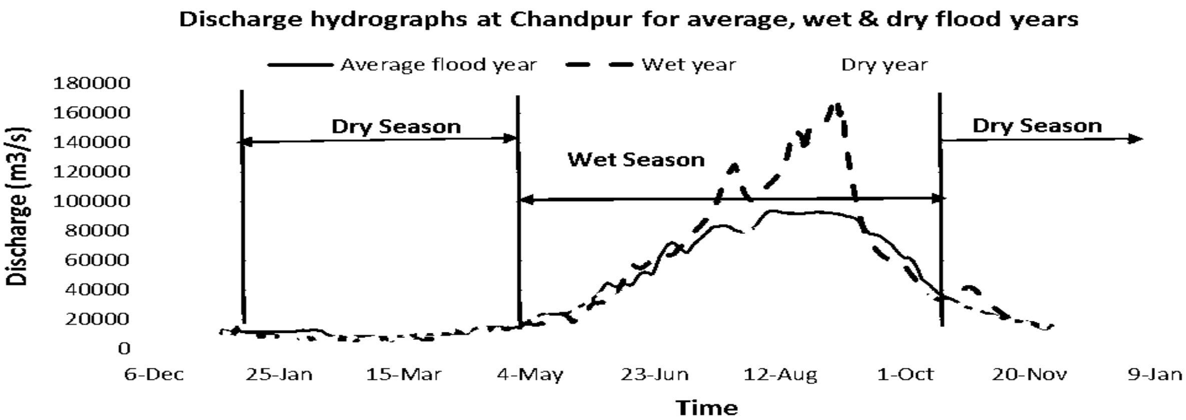

Figure 6: Computed discharge hydrographs at Chandpur for average, wet and dry flood years

Table 1: Discharge and flow volume at Chandpur of Lower Meghna for different flooding condition and different seasons

Flood Condition	Discharge m^3/s			Flow Volume E+12 m^3		
	Maximum	**Minimum**	**Mean**	**Whole Year**	**Monsoon**	**Dry Season**
Average	93,737	8,177	39,081	1.13112 **100%**	1.00131 **88.52%**	0.13 **11.47%**
Extreme Wet	170,943	3,709	45,387	1.304 **100%**	1.18852 **91.09%**	0.116 **8.91%**
Extreme Dry	84,071	2,840	31,984	0.919126 **100%**	0.81411 **88.57%**	0.105016 **11.42%**

Computed discharge hydrograph at Chandpur for the three flooding scenario is shown in Figure 6. From the discharge hydrograph the flow volume is calculated as:

$$V = \int_0^T Q dt \quad (1)$$

Where V is the flow volume, Q is the instantaneous discharge, dt is the time step of the model and T is the total duration of the hydrograph.

Computed maximum, minimum and mean discharges for different flooding scenario along with the seasonal flow volume is shown in Table 1. As expected, the minimum discharge is found to occur in the extreme dry condition, whereas, the maximum discharge is found to occur in the extreme wet condition. The extreme minimum discharge is found to be around 3000 m^3/s during the second week of March and the extreme maximum discharge is found to be around 171,000 m^3/s during the first week of September. So, in the Lower Meghna estuary, the discharge varies between 3000 m^3/s to 171,000 m^3/s considering all the possible flooding and seasonal variability. In another 1D model study comprising only the major rivers, Islam and Chowdhury (2002) found that the maximum discharge at Chandpur for the 1998 flood (which was a 100 year return period flood) was about 140,000 m^3/s.

The maximum discharge in a typical dry year is about 90% of the maximum discharge during an average flood year. So it anticipates that the other estuaries of the estuarine systems play vital role for upstream fresh water flow during extreme dry condition. Also, the tide dominance will be greater in this case due to reduced upstream fresh water flow during the dry season. This may increase extension the salinity intrusion over the Lower Meghna estuary.

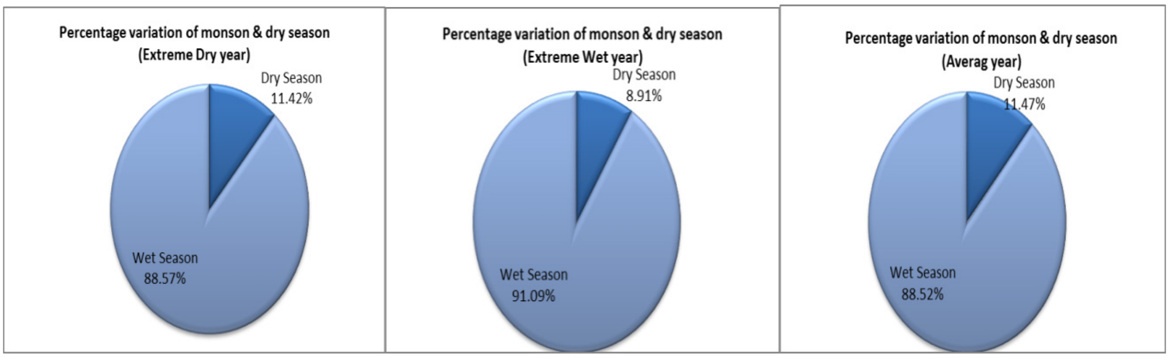

Figure 7: Pie diagram showing the percentage variation of monsoon and dry season flow volume for different flooding conditions.

On an average the flow volume at Lower Meghna Estuary over a year is more than 1 trillion m^3. It increases to 1.3 trillion m^3 in an extreme flooding condition while it falls down to 0.1 trillion m^3 in extreme dry condition. So, total flow volume in the Lower Meghna estuary varies between 0.1 trillion m^3 to 1.3 trillion m^3. The minimum water is available in the driest of the dry condition and the maximum water is available in the wettest of the wet condition. It is clearly visible that (Figure 7) the flow volume during the monsoon (April-October) is much higher than during dry season (November-March) for all the flooding conditions. Both the maximum (91%) and the minimum (9%) shares are found in the extreme flooding condition. Whereas, these shares are almost the same during an average flooding and dry conditions (about 88% and 12% respectively). This shows that in terms of flow sharing during the dry season and the monsoon, the Lower Meghna estuary behaves in an ‘extreme’ manner during the wet year compared to the dry and average years.

6. CONCLUSION

The discharge in the Lower Meghna estuary varies between 3000 m^3/s to 170,000 m^3/s, covering all possibilities of seasonal and flooding scenario. The corresponding flow volume varies between 0.1 trillion m^3 to 1.3 trillion m^3. The extreme share of flow between the monsoon and the dry season occurs during the wet flooding condition rather than during the dry or average flooding condition. The maximum share of total flow during the monsoon is 91% and the minimum share of total flow is 9% during the dry season in a hydrological year.

ACKNOWLEDGEMENTS

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REFERENCES

Chowdhury, J.U. and Haque, A. (1988), Numerical Simulation of Tides and Saline Water Intrusion in the Meghna Delta, IFCRD, BUET, May 1988.

Coleman, J.A. (1969). Brahmaputra River: channel processes and sedimentation, Sedimentary Geology. 3, 129-239.

HEC RAS (2010), HEC RAS river analysis system, User's Manual, Version 4.1, US Army Corps of Engineers, Hydraulic Engineering Center, 2010.

Hussain, M.A., Hossain, M.A., and Haque A. (2009), Seasonal Variation Of Residual Currents in the Meghna Estuary of Bangladesh, Coastal Dynamics 2009, Paper No. 30.

Islam, Saiful and Chowdhury, J.U. (2002), Hydrological Characteristics of the 1998 Flood in Major Rivers,Engineering Concerns of Flood, Ed. M. Ashraf Ali et al., pp. 227-239, BUET, Dhaka, August 2002.

MES (2001), Meghna Estuary Study, Hydro-Morphological Dynamics of the Meghna Estuary**,** DHV CONSULTANTS BV, June 2001

Rahman, M., Haque, A., Siddique, M.K.B, Ali, M.R., Ahmed, M.H., Nicholls, R.J, Stephen Darby, S., Wolf, J., Sarker, M.H., Alam, S., Ahmed, I., Sumaiya,, Hossain, M.A.R., Ahmed, M., Bricheno, L.M., Torres, R., Chowdhury, S.M., Ghosh, T. (2013), A Preliminary Assessment of the Impact of Fluvio-Tidal Regime on Ganges-Brahmaputra-Meghna Delta and its Impact on the Ecosystem Resources, International Conference on Climate Change Impact and Adaptation (I3CIA-2013), Center for Climate Change and Sustainability Research (3CSR), Department of Civil Engineering, DUET - Gazipur, Bangladesh.

HYDRO-MORPHOLOGICAL CHANGE NEAR OLD BRAHMAPUTRA OFFTAKE

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ABSTRACT

In Bangladesh, most of the offtakes are getting silted due to the adverse geographical position and geometrical condition. In this study, a part of Old Brahmaputra from its offtake to Mymensingh station has been considered for hydro-morphological analyses. The Old Brahmaputra River is one of the major distributaries of the Jamuna River which becomes dying due to serious siltation of sediments in the vicinity of offtake. In addition, the intense char movements of Jamuna induce the difficulty to locate the position of mouth of offtake. To study the hydro-morphological behavior of Old Brahmaputra River different types of analyses have been carried out that include historical discharge and water level analyses, bankline shifting, dynamic behavior of offtake, sediment transport and longitudinal bed profile analyses. It is found from the analyses that annual average flows gradually reduce from 800 m^3/s in 1965 to 500-600 m^3/s in the early nineties. The maximum diversion ratio also shows a decreasing trend over the years from 1965 to 2011. During lean period, this flow diversion is significantly reduced to below 1% in the last few decades. The bankline of Old Brahmaputra River near mouth of offtake shows that drastically shifting has occurred from year to year and it didn't follow any trend. In this study, a sediment rating curve for Old Brahmaputra River is also developed by analyzing the sediment data.

Keywords: *Offtake, siltation, Old Brahmaputra, Morphology, Bankline shifting.*

1. INTRODUCTION

Most of the distributaries of major river system of Bangladesh have been silted up. The Old Brahmaputra River is one of them which have suffered from huge deposition at its offtake. The distribution of discharge and sediment transport at river offtakes is a key factor for the long term morphological development of the main rivers (Coleman, 1969). In this study, the hydro-morphological behavior of Old Brahmaputra River near offtake area has been discussed. The Old Brahmaputra River was the previous course of Brahmaputra-Jamuna River about 200 years ago (Thorne et al, 1993). Presently the Old Brahmaputra River is a distributary of the Brahmaputra-Jamuna River and flowing over the old bed of the Brahmaputra-Jamuna River (EGIS, 2000) over a large area of North Central region of Bangladesh. The old course of the Brahmaputra River, presently known as the Old Brahmaputra, takes off at Kholabarichar, approximately 10km upstream from Bahadurabad, and follows a south-easterly course via Mymensingh and Toke up to Bhairab Bazar - at the confluence with the Upper Meghna River. The total river length between the off-take and outfall is approximately 240 km (BWDB, 2005). The Old Brahmaputra River is at present reduced to a left bank spill channel of the Brahmaputra River and only active during the high stage of the Brahmaputra River. The discharge and sediment transport through the river is dependent on opening of the offtake with the Brahmaputra River. The 240km long Old Brahmaputra River is meandering in nature and has limited capacity for passing flood discharges. During the flood period, there is a tendency of channels to overflow towards the floodplains in the lower areas. Recorded highest flood level at Mymensingh station is 13.01 mPWD which occurred in 1998.The average sinuosity of the channel is 1.29 (EGIS, 2000). The average bed slope is about 8.4cm/km near Jamalpur to 5.8cm/km near Toke (FAP 3, 1993). The average grain size of the river varies between 0.005mm to 0.356mm (FAP 24, 1996).The tributary, Jinjirum coming from across the Indo-Bangladesh border joins the Old Brahmaputra River very close to its offtake and act as the main source of dry season flow of the river Old Brahmaputra. During lean period, the mouth of Old Brahmaputra River becomes dying due to serious siltation of sediments in the vicinity of the offtake. In addition, the position of offtake is in such adverse location due to the intense char movement of Jamuna, that it is very difficult to identify the mouth of offtake.

2. STUDY AREA

The study area covers the Old Brahmaputra River from its offtake at Phulchari, Gaibandha to the Mymensingh station. Figure 1 shows the location of the study area of Old Brahmaputra River. Within this study reach, there were few tributaries and distributaries, such as Jinjiram, Jhenai, Bangshi, Sirkhali and Sutia. However, some of them are not connected with the Old Brahmaputra River now. The climate of the study area is tropical monsoon. The general soil types of the area are Non-calcareous gray flood plain soils. The landuse / land cover of the area is categorized in to cultivated land, forest, plantation and barren land (Alam et al, 2007).

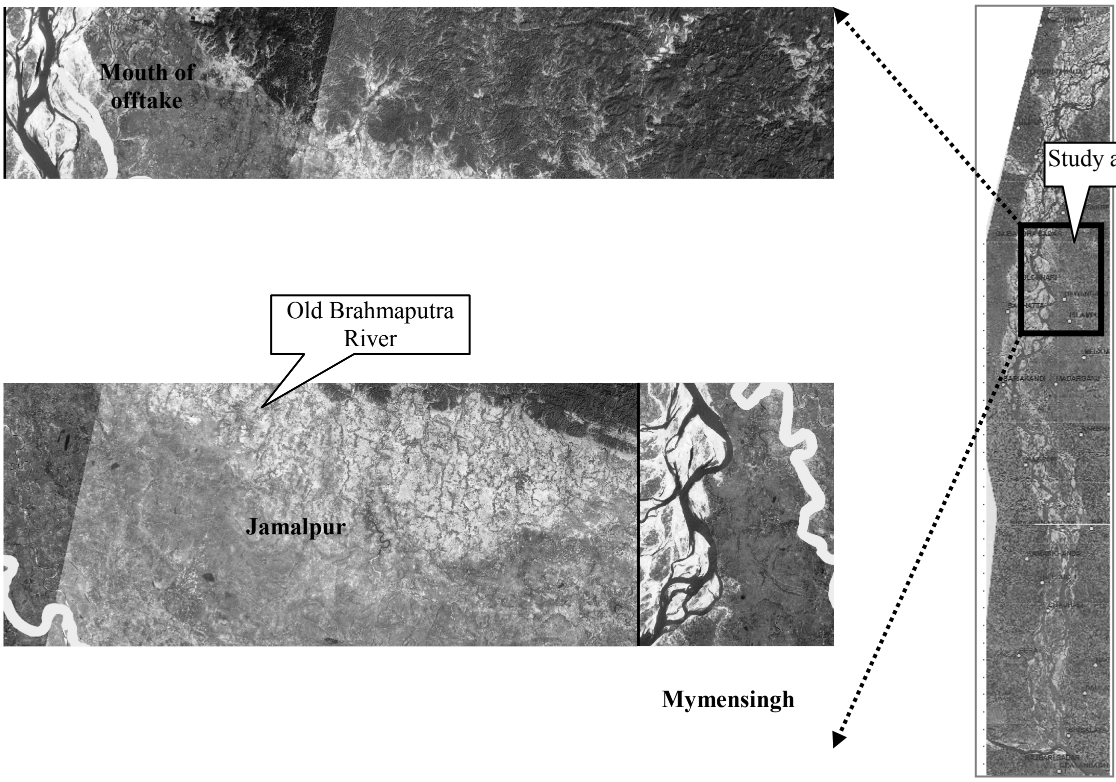

Figure 1: Study area of Old Brahmaputra River

3. DATA COLLECTION AND METHODOLOGY

For hydrological and morphological analysis different types of relevant data have been collected from various sources. Water level, discharge, sediment data and cross-sectional data have been collected from BWDB, satellite images and other information were collected from IWM. Based on the historical water level and discharge data, an extensive analysis has been done to understand the hydrological condition of the study area. On the other hand, morphological analysis such as planform analysis, bankline shifting, shifting of mouth of offtake etc have been carried out by using different satellite images with the aid of GIS tool. Moreover, longitudinal bed profile has also been conducted to observe the bed level condition and thalweg of the river.

4. RESULTS AND DISCUSSIONS

4.1 Hydrological Analysis

Historical water level data at Bahadurabad and Jamalpur station have been analysed that exhibits the water level is in declining trend in Old Brahmaputra River since 1960 but in Jamuna the water level did not show any trend, Figure 2 and Figure 3. Likewise, historical discharge data of Jamuna River at Bahadurabad station suggest that the maximum and minimum discharges have been occurred during 1998 and 2001 respectively. Otherwise there is no dramatic change of discharge in Jamuna River, Figure 4. On the other hand, the available historical discharge data of Old Brahmaputra at Mymensingh station shows that the peak discharge of Old Brahmaputra River is continuously decreasing from 1964, Figure 5. The annual flow of the river varies substantially as found from the data of 1964 to 2011 where the highest recorded discharge of the river at Mymensingh is 4890m^3/s during monsoon (September 1, 1988) and the lowest recorded discharge at the same station was 12m^3/s during the dry period. Rivers that experience such large fluctuation of discharge tend to be unstable and the river may change its morphology in medium to long-term perspective. At Mymensingh maximum monthly flow is about 2900 m^3/s. Over the years the annual flows have gradually reduced from about 800 m^3/s in 1965 to 500-600 m^3/s in the early nineties, which is equivalent to respectively 4% and 3% of the average Jamuna river discharge. The annual flow volume during the period of 1973-97 showed a clear declining trend and it appears the annual flow volume of the river has reduced to 50% since 1973, Figure 6 (EGIS, 2000).

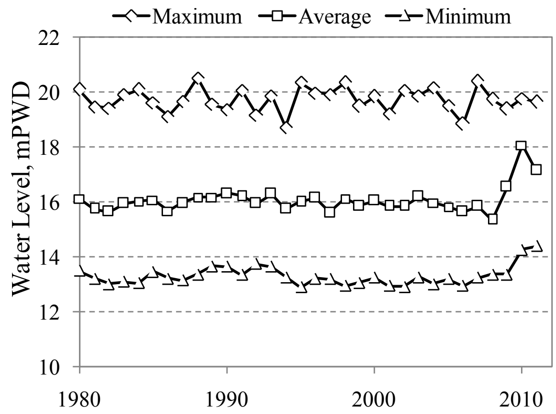

Figure 2: Maximum, minimum and average water level at Bahadurabad station of Jamuna River

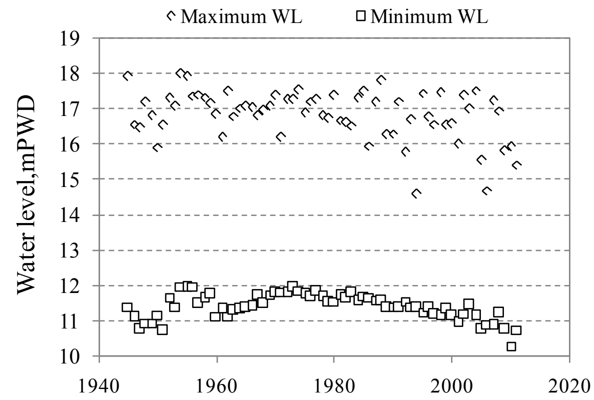

Figure 3: Maximum and minimum water level at Jamalpur station of Old Brahmaputra River

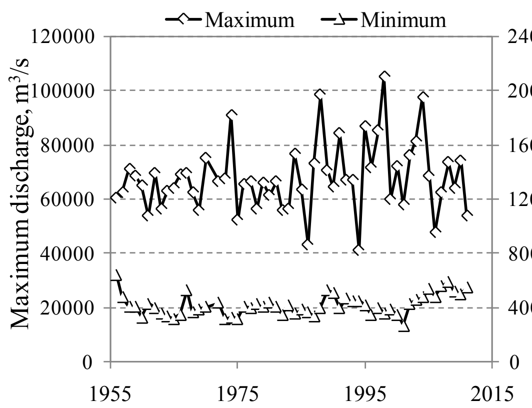

Figure 4: Maximum and minimum discharge at Bahadurabad station of Jamuna River

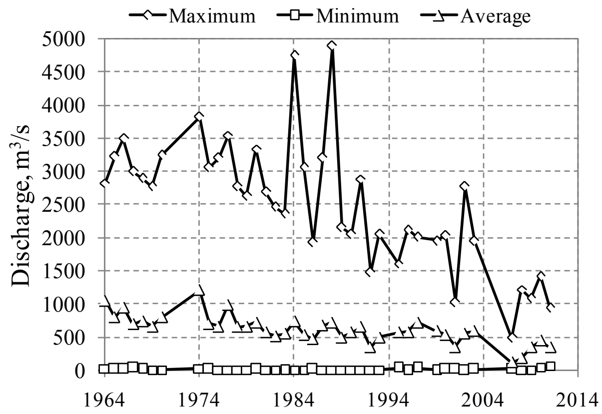

Figure 5: Discharge variation at Mymensingh station in the Old Brahmaputra River

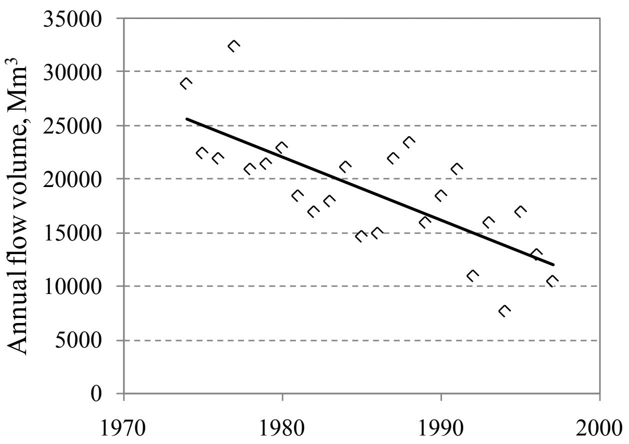

Figure 6: Changes of annual flow volume of the Old Brahmaputra River at Mymensingh (EGIS, 2000)

The percentage of maximum flow through Old Brahmaputra also shows a decreasing trend, shown in Figure 8. The value has been reduced from 4.48 % to 1.74 % during 1964 to 2011. This reduction of flow reveals the siltation problem at the mouth. From Figure 9, it is seen that percentage of flow during dry period was decreasing year to year. In 1964-71, the percentage of flow was about 5% in October, whereas it is less than 1% during 2003-11.

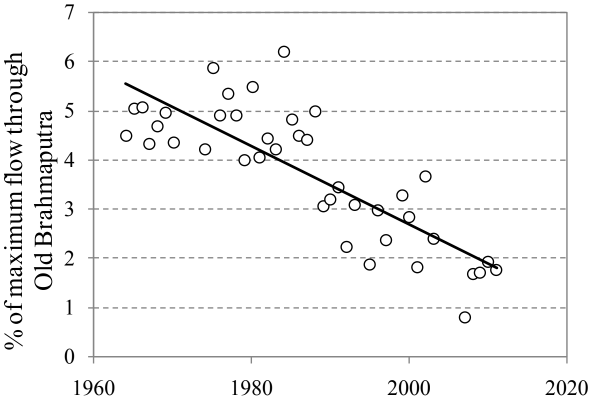

Figure 8: Percentage of maximum flow through Old Brahmaputra River

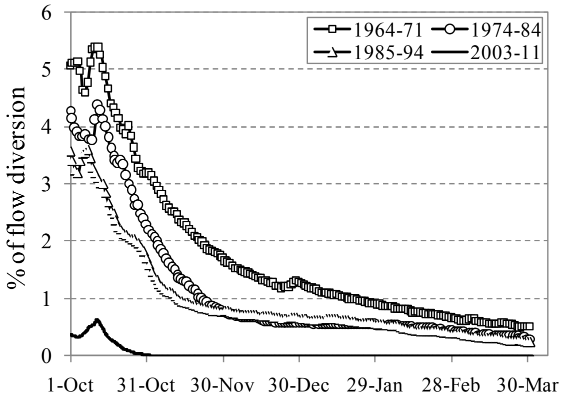

Figure 9: Percentage of flow diversion during dry season in different period

Sediment transport analysis has been done for Jamuna River by using the available suspended sediment data during the period from 1968 to 2011 using BWDB data. The annual average sediment transport was found 715 million tons during 1995 in Jamuna River whereas it was 50 million tons in Old Brahmaputra River, Figure 10 and 11. According to FAP 24 data, the annual average sediment transport was about 590 million tons in Jamuna River (FAP 24b, 1996). The maximum sediment transport was observed in 1990 (760 million tons per year).

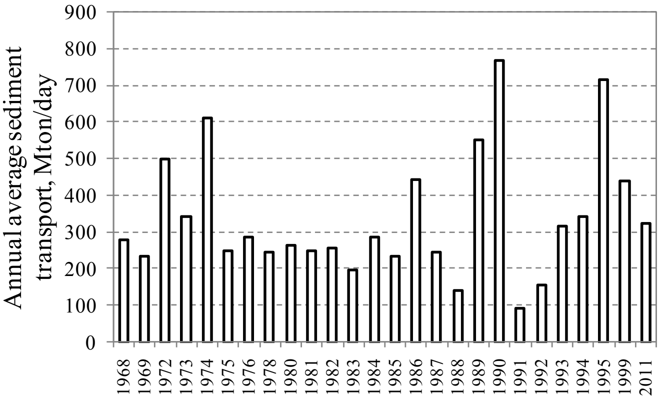

Figure 10: Sediment transport of Jamuna River

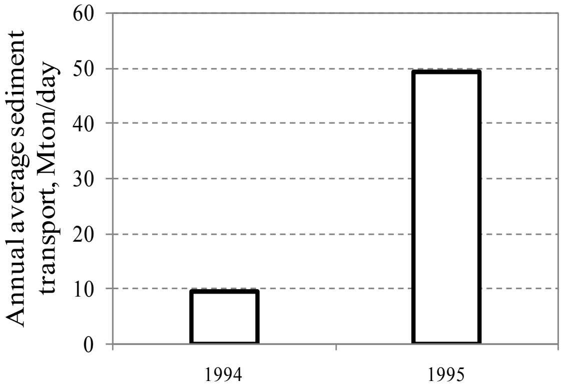

Figure 11: Sediment transport of Old Brahmaputra River

Using the available sediment discharge data of Bahadurabad station and Mymensingh station, sediment rating curve of Jamuna and Old Brahmaputra have been prepared and represented in Figure 12 and Figure 13. From trend line analysis the following relationships have been found.
For Jamuna River the equation is as follows:

$$Q_s = 0.133\ Q^{1.474} \qquad (1)$$

and for Old Brahmaputra River the equation is as follows:

$$Q_s = 0.052\ Q^{2.029} \qquad (2)$$

Where, Q_s and Q is the sediment discharge and flow discharge

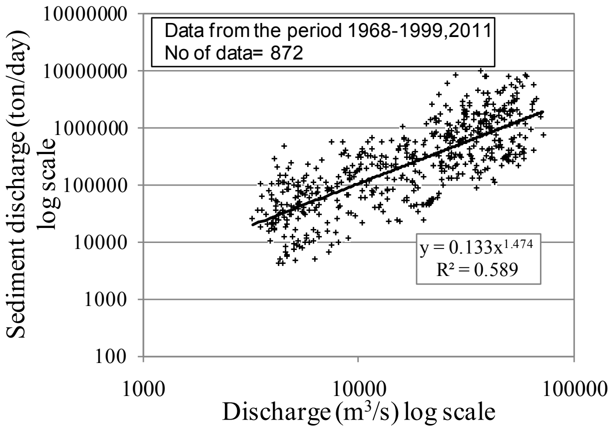

Figure 12: Sediment rating curve of Jamuna River at Bahadurabad station during 1968-1999, 2011

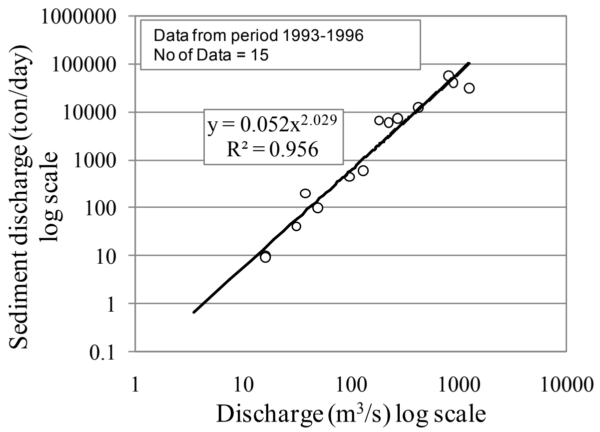

Figure 13: Sediment rating curve of Old Brahmaputra River at Mymensingh station during 1993-1996

4.2 Morphological Analysis

Morphological analyses at the study area have been carried out using available satellite images. The planform analysis illustrated the rapid morphological changes around the offtake area. Hence, a representative satellite image from every decade has been shown in Figure 14. It is seen from the figure that in 1973 the main channel of Jamuna River near Old Brahmaputra reach touched along the left bank and at that time the mouth of the offtake was apparent. Moreover, Jinjiram River was also contributing some flow to the Old Brahmaputra River during that period. In 1984, the main channel of Jamuna near offtake area was getting anabranched, though the left anabranch touched the mouth of offtake. Satellite image showed that Jinjiram River still contributes certain amount of flow to the Old Brahmaputra River. In 1997, the char near the offtake (shown in 1984 image) became attached with the left bank and a very narrow channel shared some flow to the Old Brahmaputra River from Jamuna. The major portion of flow was passed from the Jinjiram River. The recent image of 2011 shows that the attached char along the offtake has separated from the left bank and a narrow channel is flowing near the offtake which distributes certain amount of flow to the offtake.

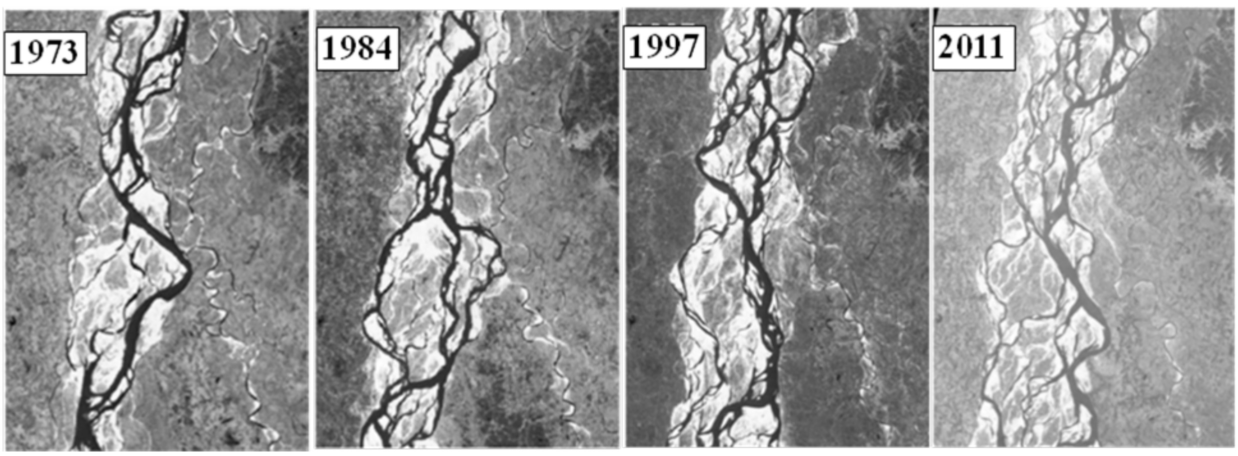

Figure 14: Satellite images in study area showing the offtake of the Old Brahmaputra River in different period.

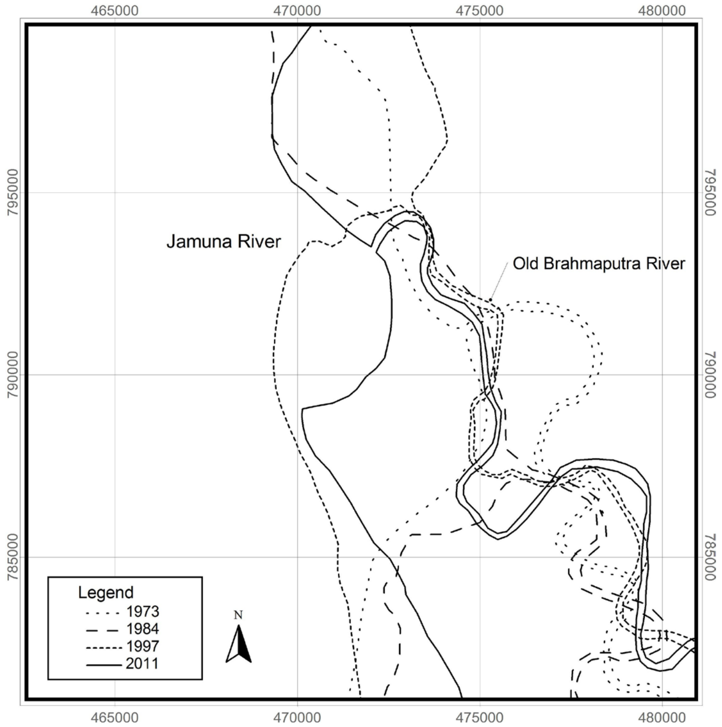

Figure 15: Location of the offtake of the Old Brahmaputra River at different time period.

Offtake of the river is the most dynamic and uncertain part of the river. In case of Old Brahmaputra, the location of offtake shifted in double magnitude from one place to the other. Moreover, in many occasions it was found that the location of the offtake was not well defined or there might be present more than one opening of a single river. The bankline shifting near Old Brahmaputra offtake shows that the river is shifting drastically from year to year and it didn't follow any trend, Figure 15. The location of offtake shifted along the left bank of the Brahmaputra-Jamuna River within 15 km stretches of the river (1973-2011), shown in Figure 16.

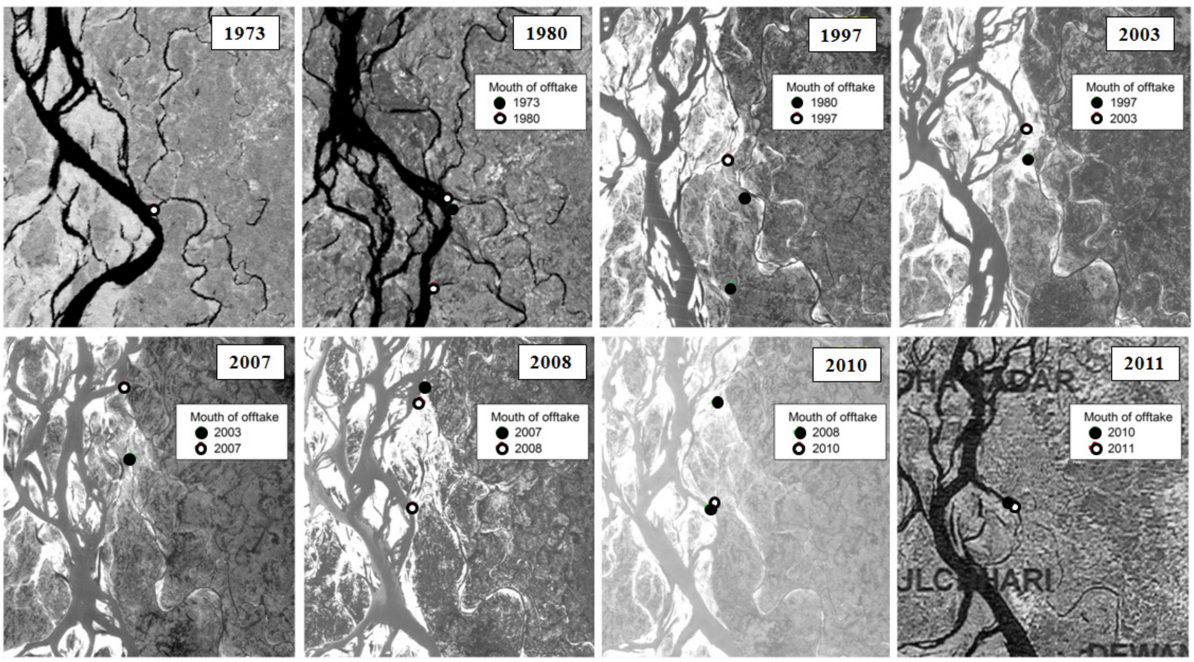

Figure 16: Changes of the offtake of Old Brahmaputra River at different time period.

The longitudinal profile of Old Brahmaputra River from offtake to Mymensingh suggested that the bed level is getting comparatively higher from the previous years (Figure 17). The bed level at offtake was around 10 mPWD during 1979-80, whereas it is near 15to16mPWD in 2011 due to huge sedimentation. The bed level slope was 7.8m/km within this reach in the period of 1979-80, however it became mild during 2011 and was about 8.4m/km.

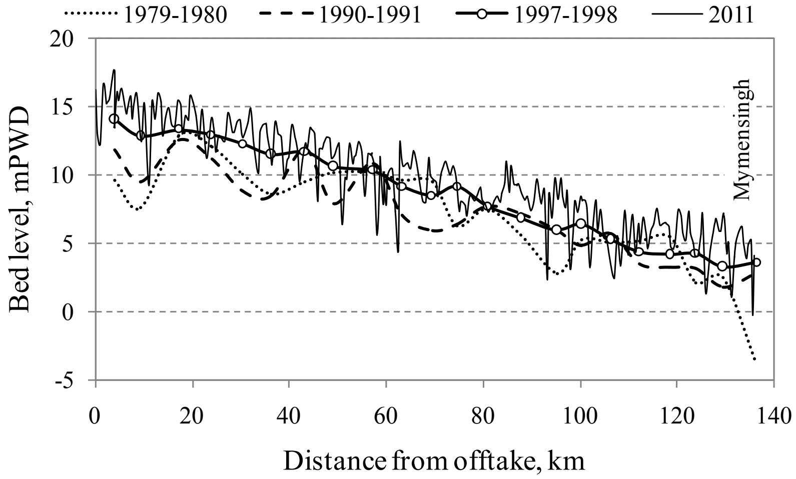

Figure17: Longitudinal profile of bed level of Old Brahmaputra River from the offtake to Mymensingh at different years.

5. CONCLUSIONS

The main Old Brahmaputra River had already been lost its conveyance capacity of water originated from the Jamuna River during the lean season due to high sedimentation in the offtake. The flow diversion during dry period is continuously decreasing in the last few decades. During 1964-74, this ratio was nearly 5% (of main Jamuna flow) at the end of monsoon which was reduced to 0.5% at the end of dry period. In the recent decade (2003-11) this values have been significantly reduced and at the end of dry period the flow diversion is almost zero. The declining pattern of Old Brahmaputra river flow indicates that sedimentation has taken place on the river bed. In addition, historical images show that the position of offtake is continuously swinging within the region of 15 km stretches of the river (1967-2011). Moreover, in many occasions it was found that the location of the offtake was not well defined or there might have been present more than one opening of a single river. Therefore, it is very difficult to identify the mouth of offtake.

REFERENCES

Alam, J.B., Uddin, M., Ahmed, J.U., Cacovean, H., Rahman, M. H., Banik, B.K. and Yesmin, N. (2007). “Study of Morphological Change of River Old Brahmaputra and it’s Social Impacts by Remote Sensing.” Geographia Technica No 2, pp. 1-11.

BWDB, (2005). “Rivers of Bangladesh.”, Published by Bangladesh Water Development Board, Bangladesh.

Coleman, J.M. (1969). “Brahmaputra River: Channel Process and Sedimentation.”, Sedimentary Geology 3, pp. 129-139.

EGIS, (2000). “Planform Analysis of the Old Brahmaputra River in Connection with the Sherpur Bridge Project” Sponsored by The Netherlands Government, Executed by Resource Analysis B.V. Delft, The Netherlands.

FAP 3, (1993). “North Central Regional Study.” Supporting Report II.1: Hydrometeorology, BCEOM, Desh Upodesh Ltd., BETS Ltd.

FAP24a, (1996).“River Survey Project, Morphological Process of Jamuna River.”, Special Report 24, GoB/FPCO, Prepared for Water Resources Planning Organization, Dhaka, 1996.

FAP24b, (1996). “River Survey Project, Sediment Rating Curves and Balances.”, Special Report 18, GoB/FPCO, Prepared for Water Resources Planning Organization, Dhaka, 1996.

Thorne, C. R., Russell, A.P.G. and Alam, M.K.. (1993), “Planform pattern and Channel Evolution of the Brahmaputra River, Bangladesh”, Published in Geological Society, London, Special Publications 1993, Vol. 75, pp. 257-276.

MACROSCOPIC TWO-PHSE FLOW MODELING OF CONCENTRATED SUSPENSIONS OF WASTE WATER USING A CONTINUUM CONSTITUTIVE EQUATION

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ABSTRACT

A finite element based macroscopic two-phase flow model has been presented. The flow of a dense suspension consisting of light, solid particle in a liquid placed between two concentric cylinders was simulated. The dynamics of a suspension is modeled by a momentum transport equation for the mixture, a continuity equation, and a transport equation for the solid phase volume fraction. Next the simulation results are compared with two dimensional NMR images of solid fraction profiles in the same suspension undergoing flow between rotating concentric cylinders with two different initial conditions to verify the numerical results. Here shear induced particle migration is significant. Under these conditions, it has been found that simulating the correct initial condition is critical to matching with the experimental results. When this is done, the model results compare well with the experiments. Liquid-solid mixtures are important in a variety of industrial fields, such as oil and gas refinement, paper manufacturing, food processing, slurry transport, and wastewater treatment.

Keywords: *Macroscopic modeling, Transport equations, Momentum transport equation, Couette flow, Shear-induced particle migration.*

1. INTRODUCTION

For Liquid-solid mixtures (suspensions) are important in a variety of industrial fields, such as oil and gas refinement, paper manufacturing, food processing and wastewater treatment. These include such application as batch sedimentation, hydraulic fracturing technology and slurry transport. Particle separation due to density differences occurs in many non-colloidal mixtures of particles and liquids, and many processing activities can benefit from knowledge of the physics of systems undergoing sedimentation or rotation. For this reason, researchers are trying to develop a modeling capability that allows to predict the flow and particle transport properties of arbitrary buoyant suspensions in complex geometries.

It is now well known that flowing suspensions of particles in a liquid have been known to exhibit particle migration even in creeping flow and in the absence of significant nonhydrodynamic or gravitational effects [1–3]. In particular, Leighton and Acrivos [2] proposed scaling arguments that identified three causes of particle migration, namely, gradients in shear rate, concentration, and relative viscosity. These arguments are the basis of a constitutive model for the evolution of particle concentration in a flowing suspension proposed by Phillips et al. [4] and referred to as the diffusive flux model. This constitutive description couples a generalized Newtonian momentum equation where the local viscosity of the suspension is dependent on the local volume fraction of solids with an evolution equation to describe the shear-induced migration of the suspended particles. Subia et al. [5] extended the diffusive flux model from viscometric to multidimensional flows using a scalar shear rate invariant to describe the shear induced migration and solved the resulting equations with the finite element method.

The term viscous resuspension was first used by Leighton and Acrivos [6] to describe the resuspension of sedimented particles due to shear flow. Complex flow and particles profiles can arise resulting from a balance of gravitational flux on the particles, which tends to lead to segregation, with shear-induced migration, which can cause remixing. A number of experimental studies have been carried out to look at the effects of viscous resuspension in pipe, channel and Couette geometries [7–11]. However, little work has been done on computational modeling of viscous resuspension. Most of the existing modeling work has been at the particle-level, which though elucidating, can be computationally intensive and difficult to apply to arbitrary geometries [12–14]. Continuum approaches have a greater chance of being useful for modeling a variety of flow fields and suspensions. However, much of the continuum modeling work has used simplified equations that are either analytically tractable or solved with rudimentary numerical methods [15]. For instance, Shauly et al. [16] modeled viscous resuspension in a polydisperse system and looked at a variety of geometries, but simplified the equations to examine one-dimensional flows only. Miskin [17; 18] model viscous resuspension in channel flows, but simplify the equations to two-dimensions and use a specialized finite difference method.

One exception is the pivotal work of Zhang and Acrivos [19] who formulated a general numerical approach to modeling multidimensional viscous resuspension with few simplifying assumptions. They used a continuum approach and extended the work of Leighton and Acrivos [2] to non-neutrally buoyant suspensions with the inclusion of a hindered settling function in the particle evolution equation and a buoyancy term in the momentum equation. They discretized the theoretical model with the finite element method and examined fully developed flow profiles in a pipe flow and obtained good agreement with experiment.

R. Rao, L. Mondy [20] examined the applicability of a continuum model of viscous resuspension and base much of their work on that of Zhang and Acrivos [19] and Phillips [4], with some significant modifications.They simulated the behavior of concentrated suspensions of large, monodisperse spheres with a Galerkin, finite element, Navier–Stokes solver into which is incorporated a continuum constitutive relationship based on the diffusive flux model but modified to allow gravity effects. Results of the model are then compared with experiment. The experiments involve using nuclear magnetic resonance (NMR) imaging to determine noninvasively the evolution of the solids-concentration profiles of suspensions as they separate when subjected to a variety of slow flows in which gravity has a substantial effect. The model was first tested on batch sedimentation to insure the correct form of the hindered settling function. Once the model was validated for settling without flow, a more complex problem was simulated that included shear as well as buoyancy effects.

This study is a CFD modeling simulating the flow of a dense suspension consisting of light, solid particles in a liquid placed between two concentric cylinders. The inner cylinder rotates while the outer is fixed. Results of the model are then compared with experimental results[20].

2. THEORETICAL DESCRIPTION

The radii of the two cylinders are 0.64 cm and 2.54 cm, respectively. The inner cylinder rotates at a steady rate of 55 rpm. With the cylinder centered at (0,0), the fluid and particle motion is small along the direction of the cylinder axes. Therefore to use a 2-dimensional model is enough. Fig. 1(a) shows the corresponding geometry.

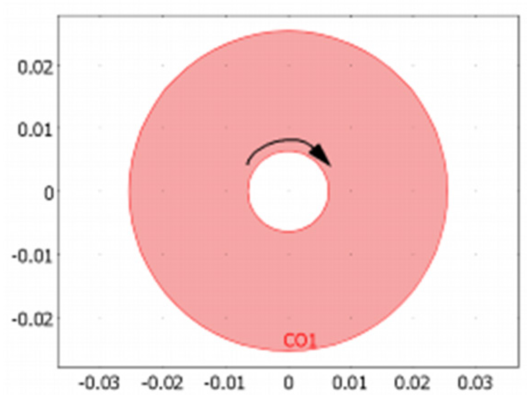

Fig. 1(a): Geometry of the Couette device. The inner cylinder rotates, the outer one is fixed.

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(i) Governing equations

The dynamics of a suspension is modeled by a momentum transport equation for the mixture, a continuity equation, and a transport equation for the solid phase volume fraction. The Mixture Model application mode automatically sets up these equations. It uses the following equation to model the momentum transport:

$$\rho\frac{\partial u}{\partial t} + \rho(\mathbf{u}.\nabla)\mathbf{u} = -\nabla p - \nabla.\left(\rho c_s(1 - c_s)\mathbf{u}_{slip}\mathbf{u}_{slip}\right) + \nabla.[\eta(\nabla\mathbf{u} + \nabla\mathbf{u}^T)] + \rho\mathbf{g} \tag{1}$$

where $\mathbf{u}$ is the mass averaged mixture velocity (m/s), ρ denotes the pressure (Pa), $\mathbf{g}$ refers to the gravity vector m/s^2. c_s is the dimensionless particle mass fraction, and $\mathbf{u}_{slip}$ gives the relative velocity between the solid and the liquid phases (m/s). Further,$\rho = (1 - \phi_s)\rho_f + \phi_s\rho_s$ is the mixture density, where $\rho_f$and ρ_s are the pure-phase densities (kg/m³) of liquid and solids, respectively, and ϕ_s is the solid-phase volume fraction (m³/m³). Finally, η represents the mixture viscosity (Ns/m²) according to the Krieger-type expression

$$\eta = \eta_f\left(1 - \frac{\phi_s}{\phi_{max}}\right)^{-2.5\phi_{max}} \quad (2)$$

where η_f is the dynamic viscosity of the pure fluid and ϕ_{max} is the maximum packing concentration.The mixture model uses the following form of the continuity equation

$$(\rho_f - \rho_s)[\nabla.(\phi_s(1-c_s)\mathbf{u}_{slip})] + \rho_f(\nabla.\mathbf{u}) = 0 \quad (3)$$

The transport equation for the solid-phase volume fraction is

$$\frac{\partial\phi_s}{\partial t} + \nabla.(\phi_s\mathbf{u}_s) = 0 \quad (4)$$

The solid-phase velocity, $\mathbf{u}_s$, is given by $\mathbf{u}_s = \mathbf{u} + (1-c_s)\mathbf{u}_{slip}$. Consequently, Equation (4) is equivalent to

$$\frac{\partial\phi_s}{\partial t} + \nabla.\left(\phi_s\mathbf{u} + \phi_s(1-c_s)\mathbf{u}_{slip}\right) = 0 \quad (5)$$

Rao and others [20] formulate the continuity equation and the particle transport in a slightly different way. Instead of the slip velocity, $\mathbf{u}_{slip}$, they define a particle flux, $\mathbf{J}_s$ (kg /(m²s)), and write the continuity equation as

$$\nabla.\mathbf{u} = \frac{\rho_s - \rho_f}{\rho_s\rho_f}(\nabla.\mathbf{J}_s) \quad (6)$$

and the solid phase transport according to

$$\frac{\partial\phi_s}{\partial t} + \nabla.(\phi_s\mathbf{u}) = -\frac{(\nabla.\mathbf{J}_s)}{\rho_s} \quad (7)$$

By comparing Equation (6) and Equation (7) with Equation (3) and Equation (5), it is clear that they are equivalent if

$$\mathbf{u}_{slip} = \frac{\mathbf{J}_s}{\phi_s\rho_s(1-c_s)} \quad (8)$$

In this model particle flux, $\mathbf{J}_s$ is used as suggested by Subia and others [5] and Rao and others [20], but the open and editable format of COMSOL Multiphysics makes it possible to specify the expression arbitrarily. Following Rao and others, the particle flux is

$$\frac{\mathbf{J}_s}{\rho_s} = [\phi D_\phi\nabla(\dot{\gamma}\phi) + \phi^2\dot{\gamma}D_\mu\nabla(\ln\mu)] + f_h\,\mathbf{u}_{st}\,\phi \quad (9)$$

Here, $\mathbf{u}_{st}$ is the settling velocity (m/s) of a single particle surrounded by fluid and D_ϕ and D_μ are empirically fitted parameters (m²) given by

$D_\phi = .41a^2$
$D_\mu = .62a^2$
where a is the particle radius (m).The shear rate tensor, $\underline{\dot{\gamma}}$ (1/s), is given by

$$\underline{\dot{\gamma}} = \nabla\mathbf{u} + (\nabla\mathbf{u})^T \quad \mathbf{(10)}$$

and its magnitude by

$$\dot{\gamma} = \sqrt{\frac{1}{2}\left(\underline{\dot{\gamma}}:\underline{\dot{\gamma}}\right)} \quad (11)$$

which for a 2-dimensional problem is

$$\dot{\gamma} = \sqrt{\frac{1}{2}\left(4u_x{}^2 + 2\left(u_y + v_x\right)^2 + 4v_y{}^2\right)} \tag{12}$$

The settling velocity, $\mathbf{u}_{st}$, for a single spherical particle surrounded by pure fluid is given by

$$\mathbf{u}_{st} = \frac{2^{a^2}}{9}\frac{(\rho_s - \rho_f)}{\eta_0}\mathbf{g} \tag{13}$$

For several particles in a fluid, the settling velocity is lower. To account for the surrounding particles, the settling velocity for a single particle is multiplied by the hindering function, f_h, defined as

$$f_h = \frac{\eta_f(1-\phi_{av})}{\eta} \tag{14}$$

where ϕ_{av} is the average solid phase volume fraction in the suspension, η_f is the dynamic viscosity of the pure fluid (Ns/m²), and η is the mixture viscosity.

(ii) Boundary conditions

The boundary conditions of the present study are described as follows.

1. There is no particle flux through the boundaries.
2. The suspension velocity satisfies no-slip conditions at all walls.
3. The fluid and particle motion is small along the direction of the cylinder axes

(iii) Initial conditions:

The Initial conditions of the present study are described as follows

1. the particles are evenly distributed within the device
2. the particles are initially gathered at the top of the device.

A commercial finite element package COMSOL Multiphysics 3.4 was used to obtain the solution. Fig.1(b) shows the surface mesh setup.Triangular quadratic Lagrange elements were employed to discretize the computational domain. As the accurateness of the numerical results depends stalwartly on the mesh sensitivity, approximately 2848 elements with minimum element quality 0.8143,1496 mesh point and element area ratio 0.114 were decided to obtain the result. Relatively fine meshes were used near the inner cylinder boundary.

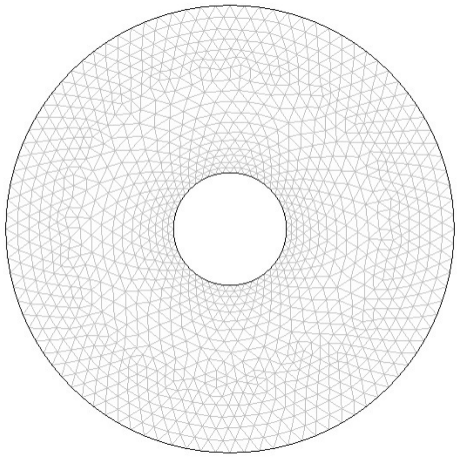

Fig. 1(b): Surface Mesh Setup

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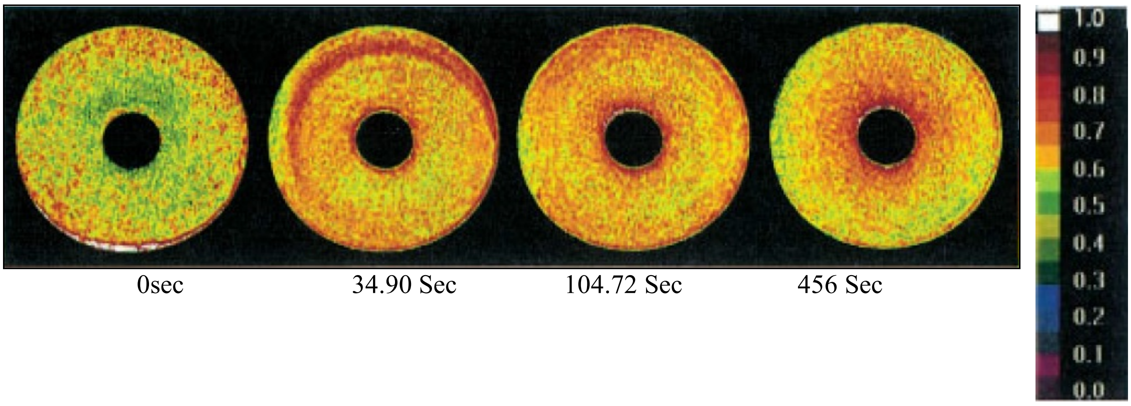
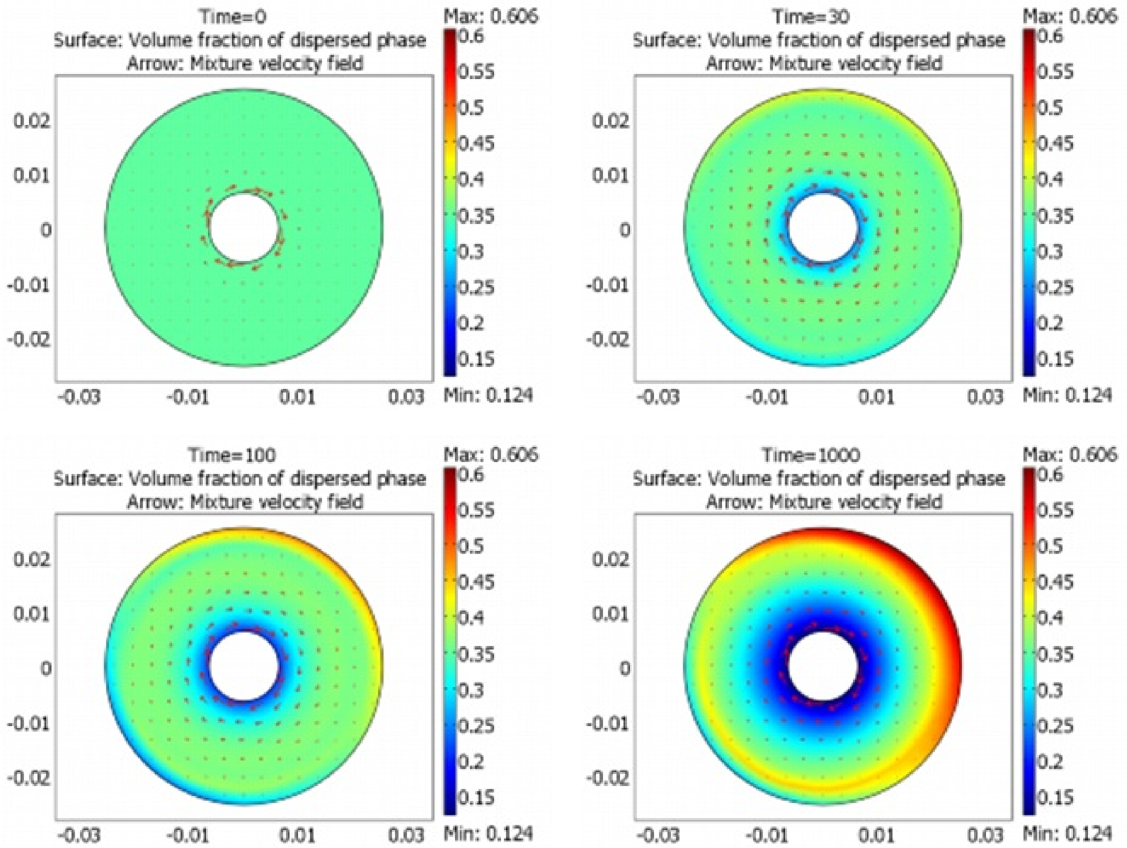
3. RESULTS

Couette flow*: *well mixed initial conditions

A comparison of experimental and finite element results for the well-mixed initial condition experiment are presented in Fig.2. Fig.2(a) depicts the NMR images of concentration profiles taken at 0, 34.90, 104.72, and 456 seconds. Even though the sample loading time has been minimized, the experimental image taken at rest reveals inhomogeneous distribution of particles prior to turning of the inner cylinder. A small, pure fluid zone at the bottom of the device has developed while zones containing as much as 50 per cent volume fraction of particles are scattered near the inner cylinder. Interestingly, the clear zone at the bottom is carried around along the streamlines towards the top of the outer cylinder and persisted even past 100 turns before viscous resuspension remixes the suspended particles in that region.

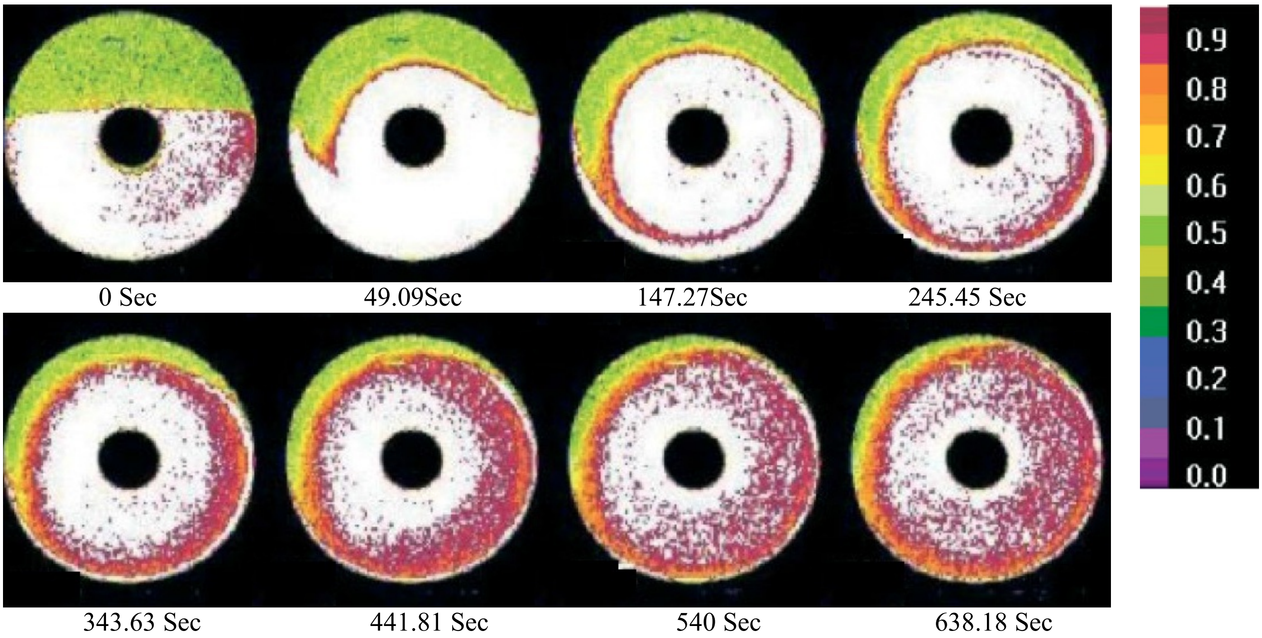


(a)

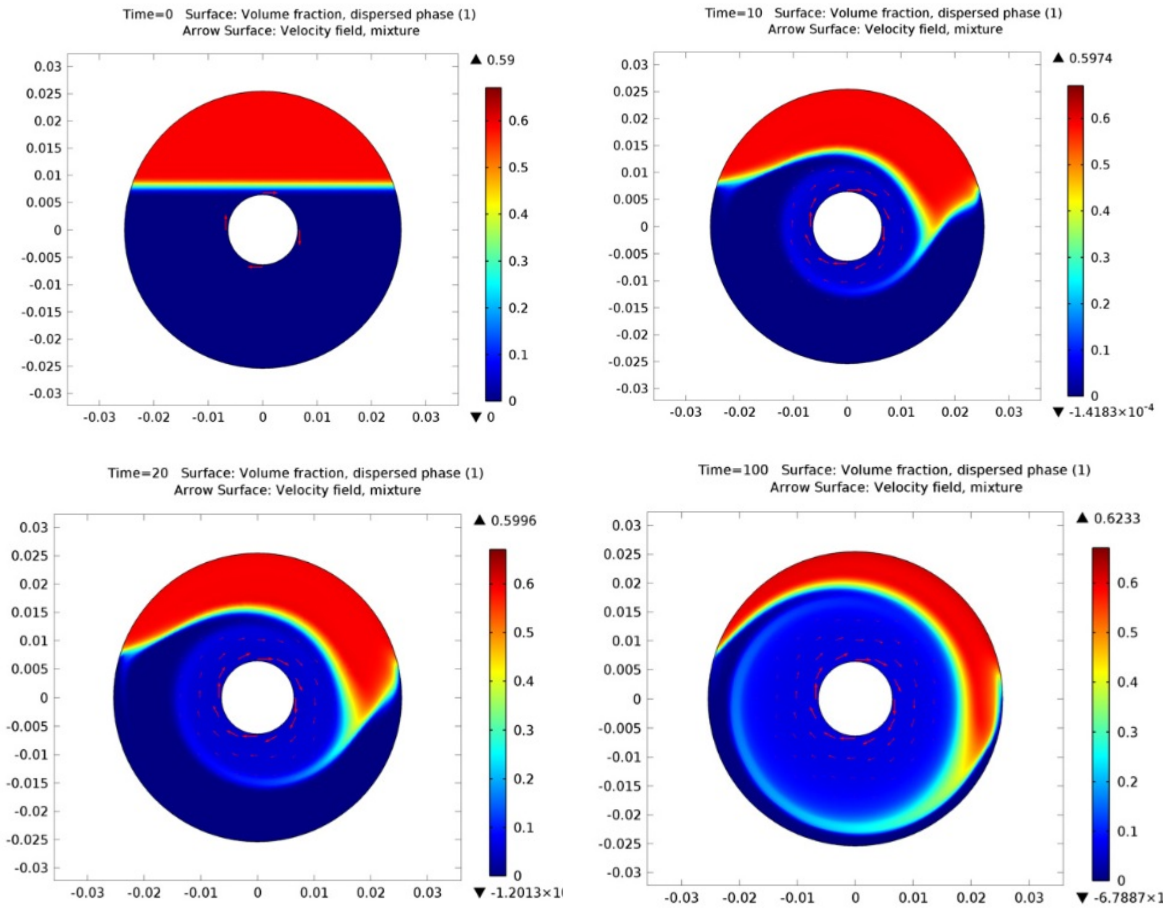


(b)

Fig.2: Couette flow concentration profiles as a function of time for initially well-mixedsuspension. (a) NMR images[20]; (b) simulation results.



(a)



(b)

Fig 3: Couette flow concentration profiles at different times. (a) NMR images, 59 per cent top, 0 per cent bottom initially[20]; (b) simulation results

The image taken at 34.90 seconds also reveal a small packed zone to the left of the low concentration band. This is probably due to accumulation of particles displaced by the low concentration band. The concentration profile at 456 seconds no longer shows signs of the initial inhomogeneity, instead it shows pronounced radial gradient particle concentration due to shear-induced migration.Snapshots of simulated particle concentration at similar time points are shown in Fig 2(b).Based on what was observed experimentally, the assumption of initially 'uniform' solution is not applied to this problem. Instead, the simulation allowed the particles to float quiescently until a clear zone with the size comparable to that observed in the experiment has developed; this consisted of about 5 min flotation without turning of the inner cylinder in both the experiment and simulation. Interestingly, when the simulations were carried out without matching the initial conditions the results matched the experiments qualitatively, but did not capture the transient correct behavior.

In the first snapshot, the model-generated profile shows different regions of low and high particle concentrations. Particles have packed near the bottom of the inner cylinder as well as along the top wall of the outer cylinder. Other than the clear zone created at the bottom, there is also a small band of low concentration particles at the top of the inner cylinder. After 30 seconds, the regions of high and low concentrations along the outer cylinder still remain, while the inhomogeneity along the inner cylinder remixes quickly as shear-induced migration becomes dominant in that region. Although the packed zone at the top of the device is not observed in the experiment, the comparison of the dynamic profiles show good qualitative agreement between the model and the experiment. At 100 seconds, the high concentration band along the outer cylinder thins out as more particles remix into the bulk. Eventually the band disappears around 175 srconds, well before the formation of another high concentration region at the top of the outer cylinder, clearly visible in that last snapshot at 1000 seconds. Such a concentrated region is not evident in the experiment, but can be viewed as a region where the suspended particles are held in place as its flotation rate cancels out the convective/diffusive flux.

Couette flow*: *sedimented initial condition

Fig 3 shows a comparison of the NMR and finite element results for the initially sedimented suspension. In Fig 3 (a), the NMR images for the second experiment indicate that the particles have packed to the top of the Couette gap before the motor is turned on. The concentration of the packed zone is 59 per cent, or 1 per cent over its theoretical maximum packing value. This is conceivable since particles of one size may exhibit some degree of polydispersity. Contrary to the first experiment, though, this packed zone thins out at a slower rate. This may be attributed to the inertial effect as it dominates the dynamics initially. Any mixing at higher turns occurs close to the outer cylinder wall while the region around the inner cylinder wall remains devoid of particles due to shear-induced migration.

The simulated concentration profiles in Fig 3(b) agree qualitatively with the NMR imaging results. An initial two-phase mixture moves in almost solid body rotation of the maximum packing zone. In fact, simulations that were run without particle diffusion show almost identical early-time results. However, the early mixing in the simulation occurs more quickly than in the experiment. By 246.54 second the effects of shear-induced migration can be seen in Fig 3(a). This migration retards the mixing of the outer layers and the simulation actually begins to lag the experimental results. However, qualitative features, such as the asymmetry created by buoyancy effects interacting with the turn direction, are preserved. There are a number of reasons that may have caused the discrepancies. To simulate solidlike behavior of the maximum packing zone, we set the viscosity to ramp to a very large value (to approximate infinity) as the concentration reaches maximum packing. However, the material in this zone cannot support stresses as a true solid would. Also, the resuspension mechanism, where individual particles peal off the packed zone, cannot be mimicked by a continuum equation. Potentially at two-phase model could be more successful at capturing the resuspension behavior. More accurate dynamics may also be achieved with a finer mesh and a better numerical scheme that can handle time-variant concentration discontinuities. The numerical method used has trouble capturing behavior when concentrations vary from maximum packing to pure fluid over an element. The kinematic shock and discontinuous concentration lead to numerical instabilities and oscillations in the solution.

4. CONCLUSIONS

In this study there is a good qualitative agreement between the simulations and the experiments. However, discrepancies exist between the shape of the concentration profiles for the quiescent sedimentation data and the numerical model. Numerical difficulties also occur in situations where large changes in concentration and thus viscosity occur over short distances. For this reason, the model of the shear flow in the well-mixed wide-gap Couette was better behaved than the quiescent settling and the Couette with the sedimented initial condition.

Many improvements to the model are being investigated to improve the agreement between the numerical results and the experimental data.

REFERENCES

1. Karnis A, Goldsmith HL, Mason SG. The kinetics of flowing dispersions I. Concentrated suspensions of rigid particles. *Journal of Colloid Interface Science* 1966; 22:531–533.
2. Leighton D, Acrivos A. The shear-induced migration of particles in concentrated suspensions. *Journal of Fluid Mechanics* 1987; 181:415–439.
3. Abbott JR, Tetlow N, Graham AL, Altobelli SA, Fukushima E, Mondy LA, Stephens TS. Experimental observations of particle migration in concentrated suspensions: Couette flows. *Journal of Rheology* 1991; 35:773–795.
4. Phillips RJ, Armstrong RC, Brown RA, Graham AL, Abott JR. A constitutive equation for concentrated suspensions that accounts for shear-induced particle migration. *Physics of Fluids* A 1992; 4:30–40.
5. Subia SR, Ingber MS, Mondy LA, Altobelli SA, Graham AL. Modelling of concentrated suspensions using a continuum constitutive equation. *Journal of Fluid Mechanics* 1998; 373:193–219.
6. Leighton D, Acrivos A. Viscous resuspension. *Chemical Engineering Science* 1986; 6:1377–1384.
7. Chapman BK, Leighton DT, Jr. Dynamics viscous resuspension. *International Journal of Multiphase Flow* 1991; 17:469–483.
8. Acrivos A, Mauri R, Fan X. Shear-induced resuspension in a Couette device. *International Journal of Multiphase Flow* 1993; 19:797–802.
9. Krishnan GP, Leighton DT. Dynamic viscous resuspension of bidisperse suspensions—I. Effective diffusivity.*International Journal of Multiphase Flow* 1995; 21:721.
10. Hallworth MA, Huppert HE. Abrupt transitions in high-concentration, particle-driven gravity currents. *Physics of Fluids* 1998; 10:1083.
11. Breedveld V, Van Den Ende D, Tripathi A, Acrivos A. The measurement of the shear-induced particle and fluid tracer diffusivities in concentrated suspensions by a novel method. *Journal of Fluid Mechanics* 1998;375:297–318.
12. Andrews MJ, O'Rourke PJ. The multiphase particle-in-cell (MP-PIC) method for dense particulate flows. *International Journal of Multiphase Flow* 1996; 22:379–402.
13. Hu HH. Direct simulation of flows of solid–liquid mixtures. International Journal of Multiphase Flow 1996;22:335.
14. Morris JF, Brady JF. Pressure-driven flow of a suspension: buoyancy effects. *International Journal of Multiphase Flow* 1998; 24:105–130.
15. Schainger U, Acrivos A, Zhang K. Viscous resuspension of a sediment within a laminar and strati..*International Journal of Multiphase Flow* 1990; 16:567–578.
16. Shauly A, Wachs A, Nir A. Shear-induced particle resuspension in settling polydisperse concentrated suspension.*International Journal of Multiphase Flow* 2000; 26:1–15.
17. Miskin I, Elliot E, Ingham DB, Hammond PS. Steady suspension flows into two-dimensional horizontal and inclined channels. *International Journal of Multiphase Flow* 1996; 22:1223.
18. Miskin I, Elliot E, Ingham DB, Hammond PS. The viscous resuspension of particles in an inclined rectangular fracture. *International Journal of Multiphase Flow* 1996; 22:403.
19. Zhang K, Acrivos A. Viscous resuspension in fully developed laminar pipe flows. *International Journal of Multiphase Flow* 1994; 20:579–591.
20. R. Rao, L. Mondy, A. Sun, and S. Altobelli, "A numerical and experimental study of batch sedimentation and viscous resuspension," *Int. J. Num. Methods in Fluids*, vol. 39, pp. 465–483, 2002.

HYDRAULIC AND MORPHOLOGICAL IMPACT ASSESSMENT OF CAPITAL DREDGING IN THE GANGES RIVER USING HEC-RAS

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ABSTRACT

Bangladesh is a great delta formed by the alluvial deposits of the three mighty Himalayan Rivers: the Ganges, the Brahmaputra and the Meghna. And these river systems carries probably the largest sediment discharge of all the world's rivers. The recent increase in bed levels due to the huge sediment inflow on the major river system of Bangladesh is introducing problems like widening of rivers, increased flood propensity, river bank erosion, reduction in navigability, closure of off-takes etc. Capital Dredging can play a key role in solving these problems. This study mainly applies mathematical modeling techniques to assess the impact of capital dredging in selected reaches of the Ganges river. A 116km long one-dimensional model has been set up for the river using HEC-RAS. A rating curve was developed using the observed discharge and water level data at Hardinge Bridge to generate the flow boundary of the model. The model has been hydrodynamically and morphologically calibrated using the latest availale data. The design storm event was determined by carrying out frequency analysis of the observed data. It is seen from the simulated model that although the velocity at the vicinity of the capital dredging has increased, it has negligible impact on the important structures in the river. Also very little backfilling is seen in the dredged channel with the passage of design storm event.

Keywords: *Rating Curve Development, 1D Modelling, Morphological Model, Capital Dredging, Impact assessment of Dredging*

1. INTRODUCTION

Bangaldesh is a great delta formed by the alluvial deposits of the three mighty Himalayan Rivers: the Ganges, the Brahmaputra and the Meghna and the Ganges-Brahmaputra-Meghna (GBM) river system carries probably the largest total sediment discharge of all of the world's rivers. In fact, all the major rivers of Bangladesh carry huge sediment loads from the large surrounding catchments, which have been estimated to about 1.0 to 1.1 billion tons annually. The recent increase in river bed levels due to the huge sediment inflow on the major river system of Bangladesh is introducing problems like widening of rivers, increased flood propensity, increased river bank erosion, reduction in river navigability, closure of off-takes of the distributary channels etc. Capital Dredging can play a key role in solving these problems. This study mainly indentifies the vulnerable reaches of the Ganges River and then applies mathematical modelling techniques to assess the impact of capital dredging in those vulnerable reaches.

For this study the one dimensional model was developed using HEC-RAS, Hydrologic Engineering Centers River Analysis System. HEC-RAS can perform one dimensional steady, unsteady flow river hydraulics computations. Along with steady and unsteady river hydraulic computations, sediment transport or mobile bed computation and water quality analysis can also be carried out using HEC-RAS. The sediment transport module has been used in this study to predict the morphological characteristics of the Ganges River for both present base conditions as well as for the planned dredged condition.

2. MODEL DEVELOPMENT

A 116km one-dimensional model was developed using the HEC-RAS software. The extent of the model starts from Char Bhabananda Diar near the Bangladesh-India border and ends near the Baruria Transit. The network for the river was drawn with the help of a geo-referenced image obtained using the Google-Earth software. The network geometry of the model is shown in Figure 1.

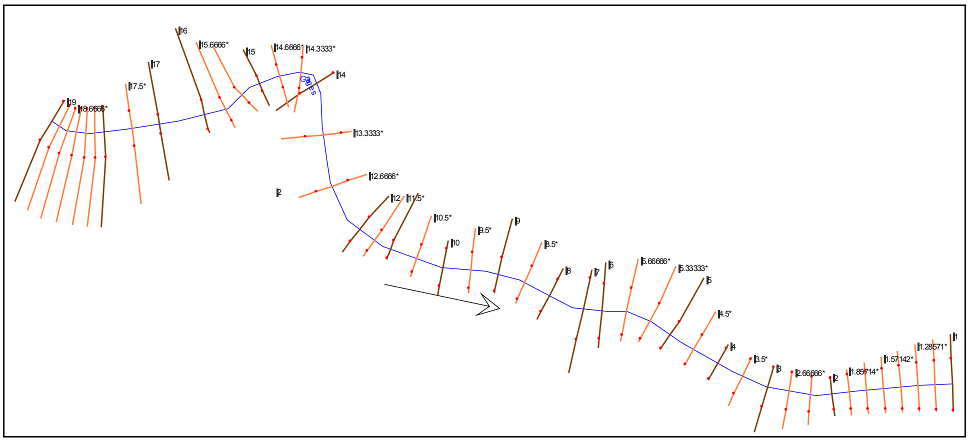

Figure 2: HEC-RAS Model Network

To simulate any model morphologically it is necessary to provide the sediment discharge boundary condition of the model. But due to the lack of enough sediment data, it was very hard to provide a time varying sediment discharge boundary. To alleviate this problem, “Equilibrium Load” condition was provided at the upstream boundary. The Equilibrium Load condition will always calculate the sediment load capacity for the given discharge and apply it in the upstream of the model. So, it will give very close result to the actual sediment for that given discharge

2.1 Rating Curve Development

The relationship between the stage and the discharge at a cross-section of the river is commonly expressed with rating curves. Generally, the process of discharge measurement is a complex procedure. But to develop models for streams and rivers, the flow is required which signifies the amount of mass entering the model system. For our study, a connection was established between the available water level data and discharge data of the Hardinge Bridge station with the aid of Microsoft Excel program.

The developed rating curve is shown in Figure 2. It is clear from the figure that a two-stage rating curve was required to fully express the stage-discharge relationship with the second stage of the rating curve starting from 9mPWD. A comparison between the observed discharge at the Hardinge Bridge and the calculated rated discharge using the developed rating curve is shown in Figure 3.

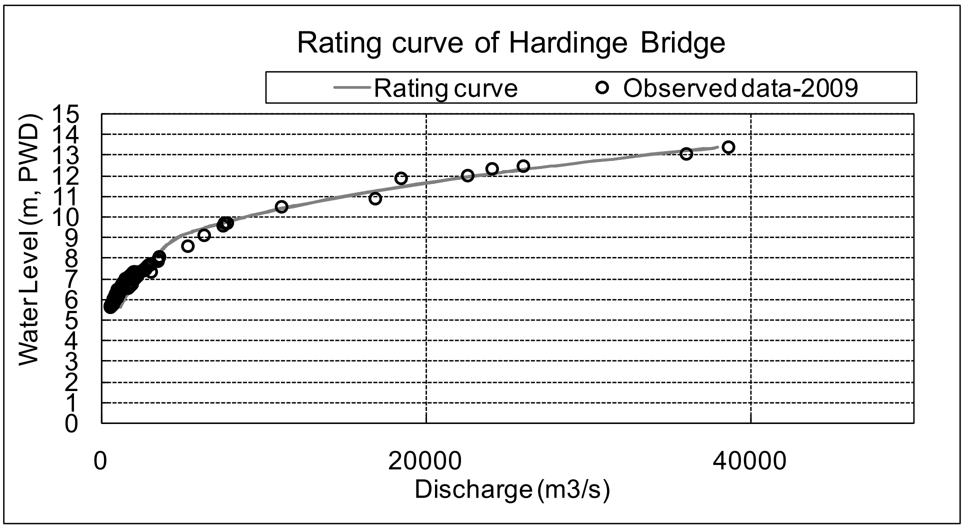

Figure 2: Stage-Discharge Rating Curve developed for this study

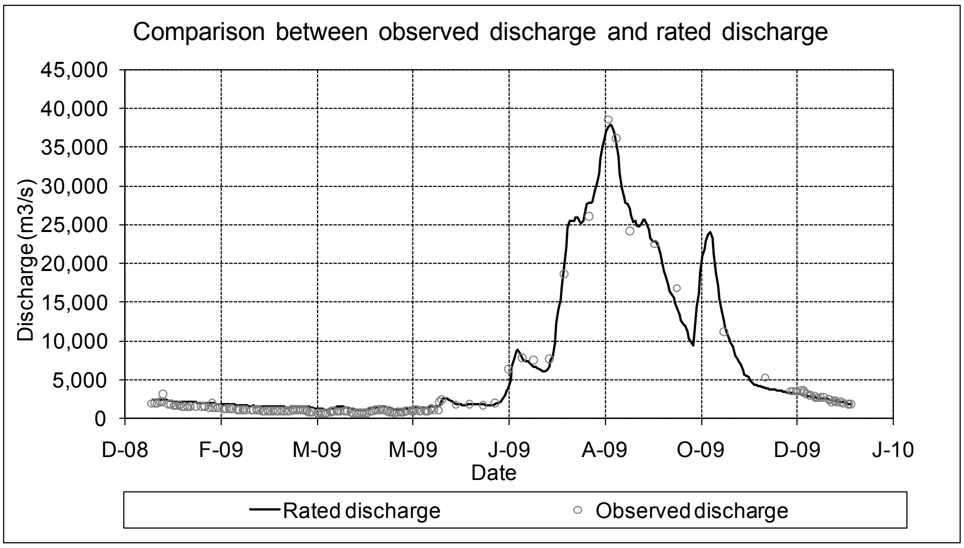

Figure 3: Comparison between the observed discharge and the rated discharge

2.2 Model Calibration

Before a model is used as a predictor, it must be calibrated against observed data. Model Calibration is an iterative process of comparing the model to actual system behaviour until model accuracy is judged to be acceptable. It is always recommended to calibrate a model for the latest available data series. For this study, both of the hydrodynamic and sediment transport model was calibrated for the 2009 hydrological event. For hydrodynamic calibration the model generated water surface elevation was extracted at Hardinge Bridge and near Sengram. The extracted data was compared with the collected observed water level data in those respective stations. The Figure 4 and Figure 5 show the hydrodynamic calibration plot for Hardinge Bridge and Sengram respectively. The Sediment Transport model was calibrated by generating the sediment rating curve with the model generated sediment load and then comparing it with the observed sediment discharge trend at Hardinge Bridge. Sediment calibration by generating sediment rating curve is shown in Figure 6.

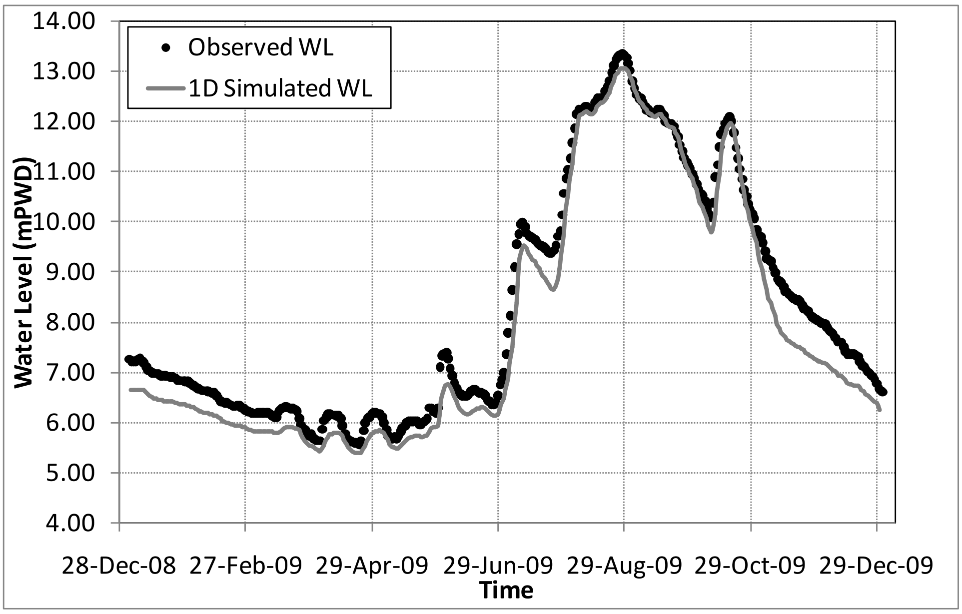

Figure 4: Water level calibration plot at Hardinge Bridge

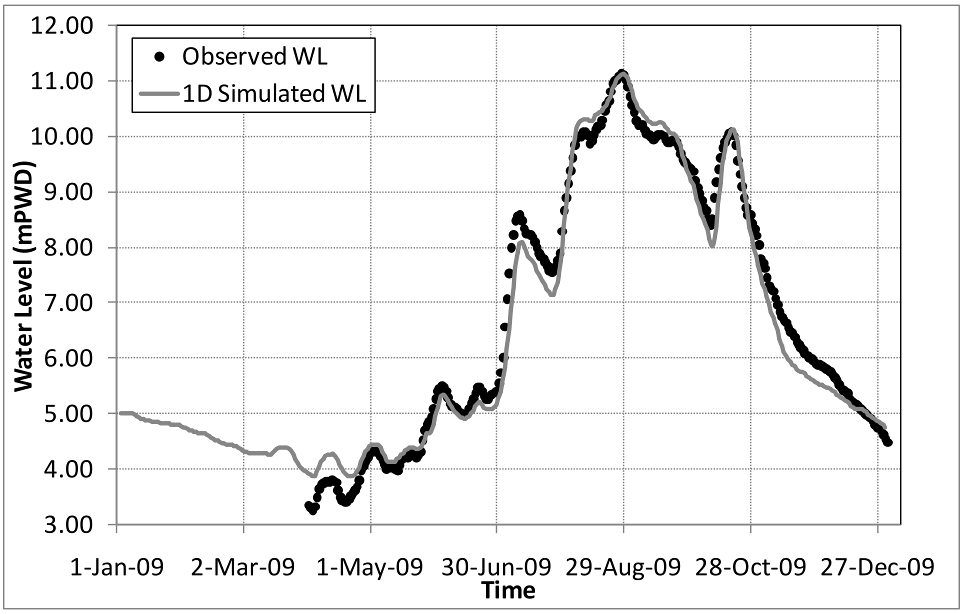

Figure 5: Water level calibration plot at Sengram

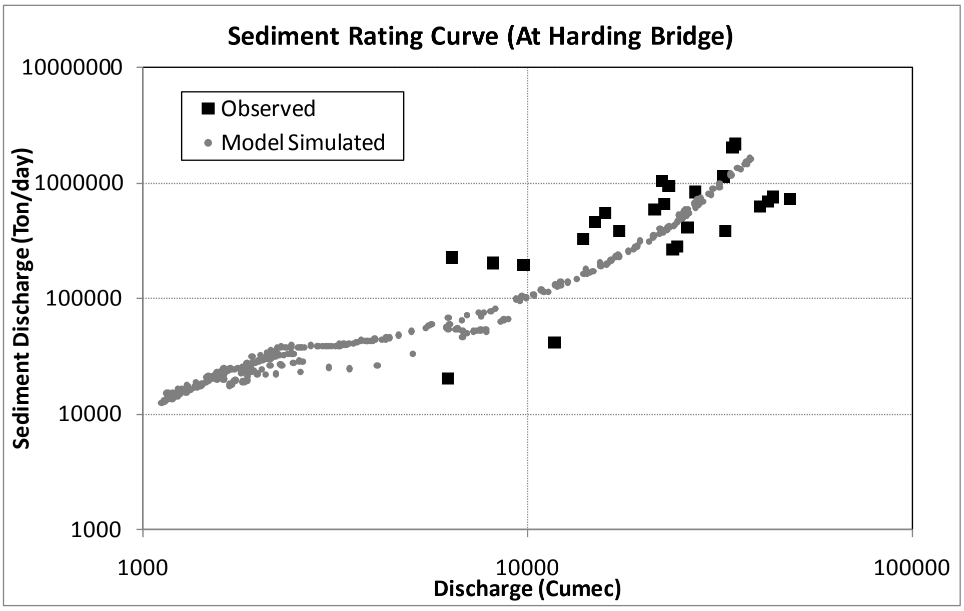

Figure 6: Sediment Calibration by developing Sediment Rating Curve

2.3 Model Validation

Model validation involves the processes of rechecking the model assumptions used during the calibration process for a different model condition. Only after a model goes through a through validation process, it can be said that the developed model is a representation of the actual system. During this study, the calibrated model parameters have been validated for two separate hydrological events. The 1998 hydrological event was chosen to validate model behaviour in extreme flood conditions. The water level validation plot of Sengram for the 1998 event is shown in Figure 7. The 2004 event was chosen to represent an average flood condition occurring in the model domain. The water level validation plot of Hardinge Bridge is for the 2004 event is shown in Figure 8.

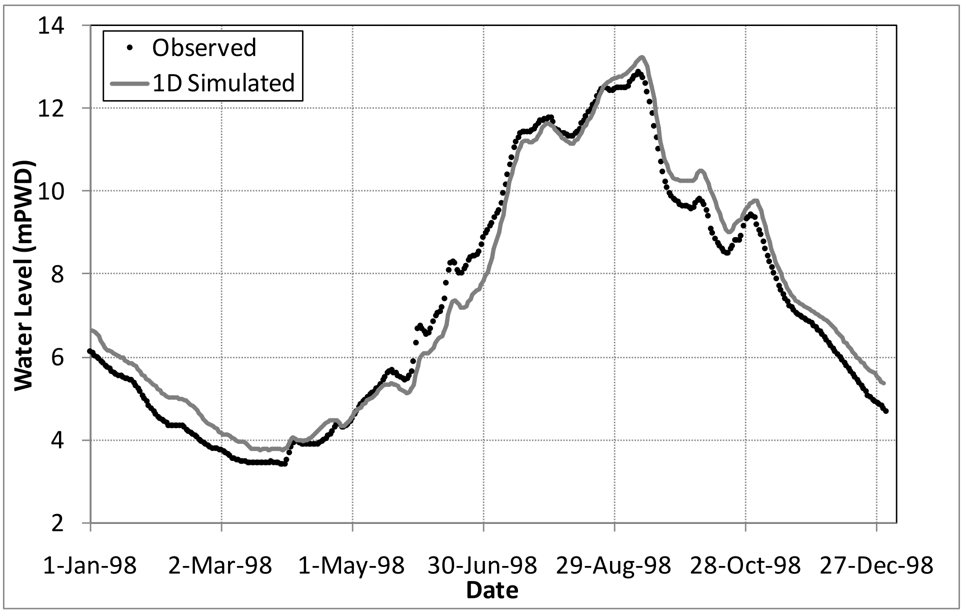

Figure 7: Water level Validation plot at Sengram for 1998

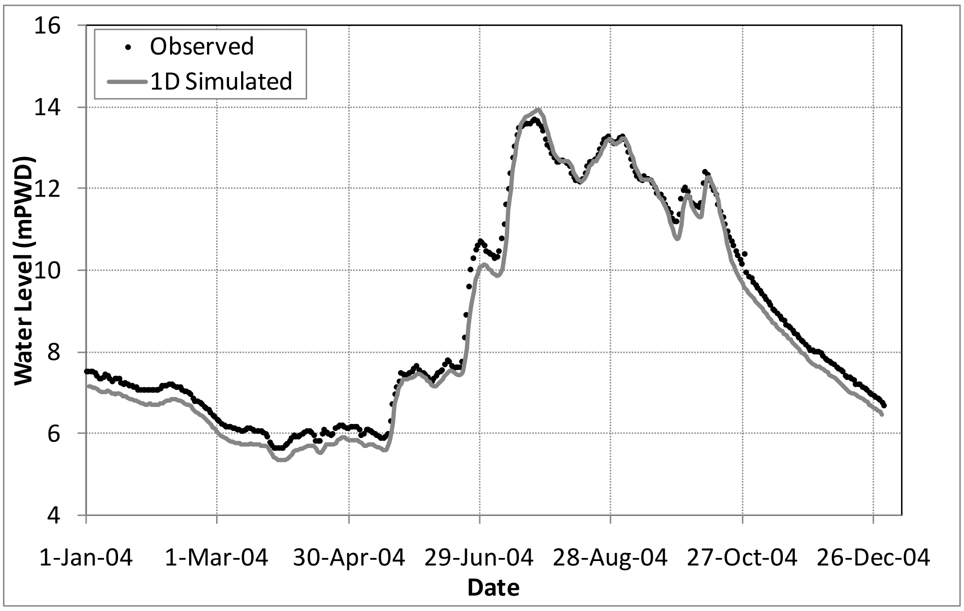

Figure 8: Water level Validation plot at Hardinge Bridge for 2004

3. DEVELOPING THE DREDGED OPTION MODEL

To develop the dredged model scenario, longitudinal profile of the Ganges River was delineated to identify the silted up area of the river. From the long profile a 38km reach starting from 16.5 km downstream of Hardinge Bridge was selected as the location for capital dredging. The average water surface slope of Ganges is around 5 cm/km and considering that slope a line was drawn in the long profile of the river taking the invert of the cross-section at Baruria transit as the base point. Figure 9 shows the drawn long profile of the Ganges River, the dredged level line inserted in the long profile and marks the area which have been planned to dredge in this study. For the purpose of this study, an arbitrary dredging channel having a width of 500m was preliminarily selected. The volume of capital dredging was extracted from the HEC-RAS result viewer.

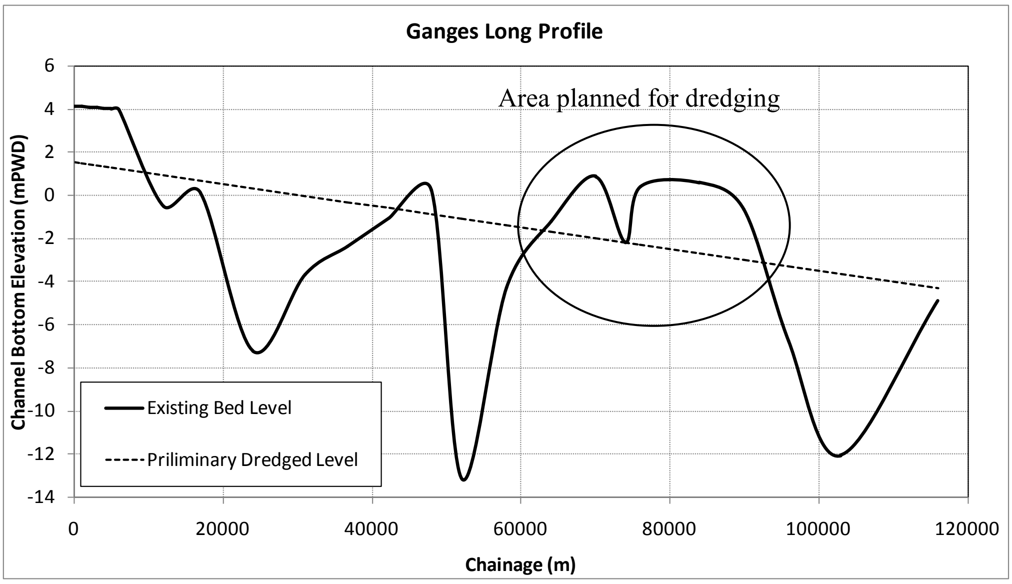

Figure 9: Long Profile of the Ganges River

3.1 Design Storm Selection Using Frequency Analysis

After selecting the dredging location and dimensions, the design storm had to be selected. Preliminarily, 100 year return period storm was selected as the design storm. And to determine the storm with 100 year return period, frequency analysis had to be carried out for the historical data using different probability distributions. For the purpose of this study, a spreadsheet program was developed using Microsoft Excel to carry out the frequency analysis portion of the study. The distribution methods incorporated in the program for the estimation of probable discharge are Gumbel, Log-Normal and Log-Pearson.

For the historical data, annual maximum discharge at the Hardinge Bridge station was used. It is worth mentioning that only the discharge data after 1976 has been considered as Farakka barrage started its operation in 1975. The final output of the conducted frequency analysis is shown in Table 1.

Table 1: Probable Discharge of the Ganges River at Hardinge Bridge

Return period	Discharge (m^3/s)		
	Gumbel	Log-Pearson	Log-Normal
2.33	52441	51368	53352
50	80124	77607	77690
100	85940	81947	82070

Fitting the discharge data in a Semi-Log graph paper we find that the Log-Normal distribution fits the best for this set of data. Considering this fact the Log-Normal distribution was chosen for determining the 100 year return period storm. The probable discharge of 1 in 100 return period storm estimated by Log-Normal method is close to magnitude peak of 1998 flood event. Thus the 1998 flood event was chosen as the design event for the scenario analysis. The Log-Normal distribution is shown on Figure 10.

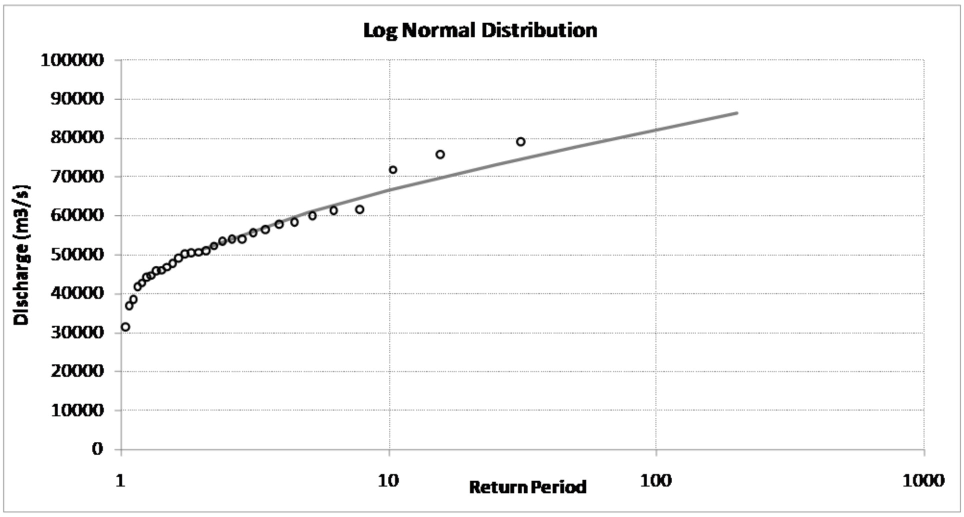

Figure 10: Log-Normal Probability Distribution for the Ganges River

3.2 Incorporating the Dredging Option in HEC-RAS

The dredging option was incorporated in the model using "Dredge Event" tool in the sediment transport module of HEC-RAS. The dredged sections were placed in such way that it required minimum amount of cutting. The dredged volume was minimized by several trial and error steps. Figure 11 show a graphical comparison of a sample cross-section of the Ganges River before and after dredging.

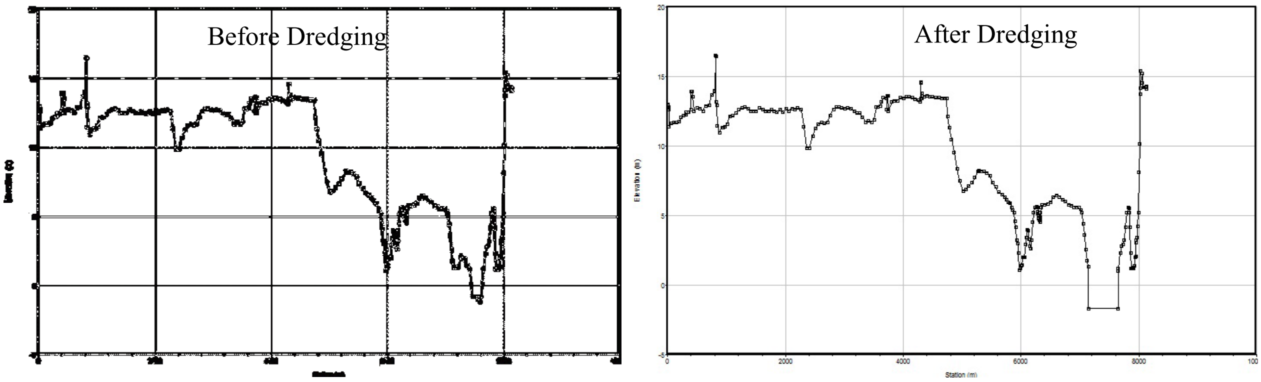

Figure 11: Cross-section comparison before and after applying dredging

4. HYDRAULIC IMPACT ASSESSMENT

Hydraulic impact assessment of capital dredging on the Ganges River on the selected locations have been analyzed on the basis of one-dimensional sediment transport model results simulated for 1 in 100 year flood event. It has been mentioned earlier (Section 3.1) that 1998 flood event has been treated as 1 in 100 year flood event. Figure 12 shows the comparison between the cross-sectional average velocities for dredged condition and base condition at Sengram. In the figure it is seen that the cross-sectional average velocity during the monsoon increases in the dredged portion of the model. However slightly different scenario is observed for the other portions of the year. The cross-sectional average velocity for the dredged option in times other than monsoon is actually lower than the base condition. The increase of conveyance area for lower water levels can be identified as the reason behind this scenario. But for the monsoon period, the conveyance area for the main channel more or less remains the same during the base and dredged conditions. But the water depth in the dredged condition is greater than in the base condition which leads to lower resistance and higher velocity.

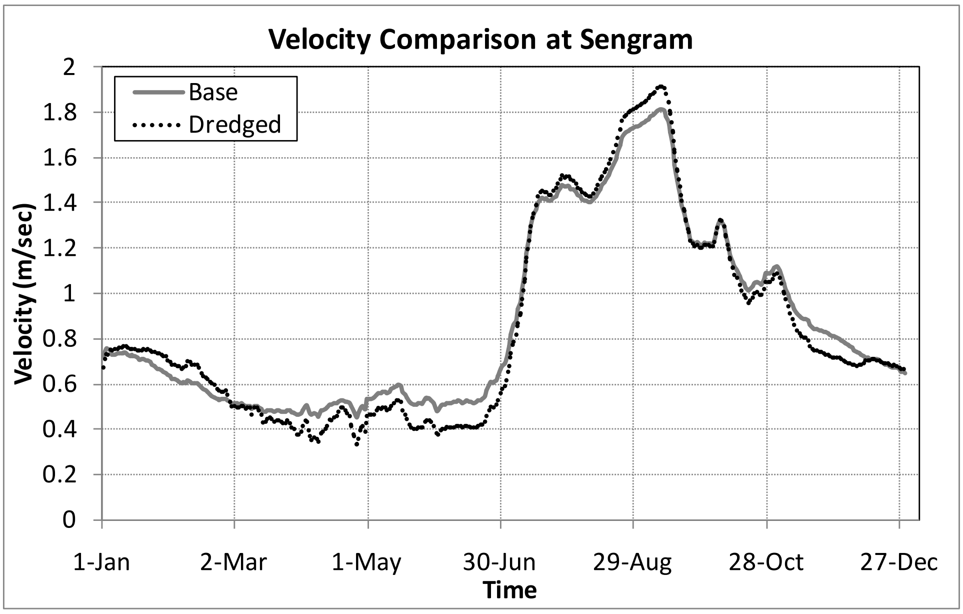

Figure 12: Cross-sectional average velocity comparison between Base and dredged condition at Sengram for the design hydrological event

No important structure exists in the planned dredging area. The closest important structure is the Hardinge Bridge and velocity comparison was also made at the Hardinge Bridge to check the effect of dredging at Hardinge Bridge which is shown in Figure 13. From the figure we can clearly see that the capital dredging has very negligible effect on the cross-sectional average velocity at Hardinge Bridge.

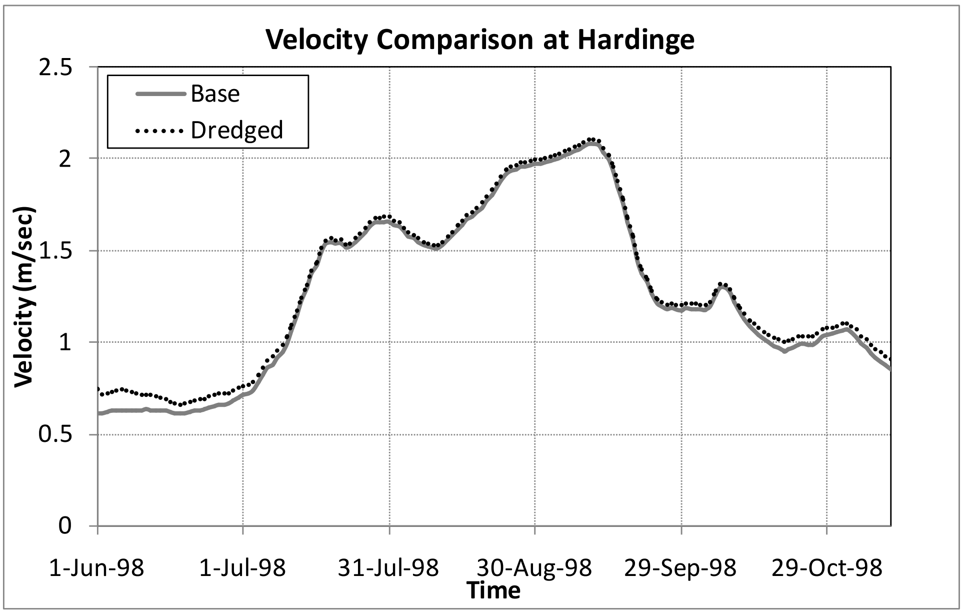

Figure 13: Cross-sectional average velocity comparison between Base and dredged condition at Hardinge Bridge for the design hydrological event

5. MORPHOLOGICAL IMPACT ASSESSMENT

In this study, the morphological sustainability of the dredged channel has been assessed. The volume of capital dredging was determined by using the result viewer of HEC-RAS. The dredged volume of all the cross-section was extracted separately and then added to find the cumulative capital dredged volume. For determining the backfilling volume, the change of bed level for one year of simulation was extracted. The change of bed level was multiplied by the dredging width and dredging length to find the backfilling volume. Table 2 shows the dredged volume and filled volume of each sections as well as the backfilling rate.

Table 2: Dredged Volume and filled volume of each of the section

Cross-Section	Capital Dredging Volume (million m^3)	Backfilling Volume (million m^3)
G-9	3.48	0.12
G-8.5	4.51	0.12
G-8	7.26	0.25
G-7	5.71	0.10
G-6	4.89	0.00
G-5.66	5.20	0.00
G-5.33	5.09	0.00
G-5	5.92	0.10
G-4.5	5.86	0.00
G-4	5.50	0.00
Cumulative	53.42	0.69
Backfilling Rate (%)		1.30

The table indicates very little siltation in the dredged channel for a 1 in 100 flood event. Thus from the model result it can be predicted that the dredging event planned for this study may sustain the design flood event without much siltation.

6. CONCLUSIONS

6.1 Findings

From analyzing the result files for capital dredging option, the following findings were found:

- The capital dredging option tested for this study will have negligible impact on the important structures located on Ganges (i.e. Hardinge Bridge)
- From the simulation of 1 in 100 flood event, it was found that the back filling rate was also very negligible. Hence we can conclude that the dredging section will sustain the design flood event.

6.2 Limitations of the Study

Morphological changes in rivers are complex processes. The accuracy of model prediction therefore is dependent on the availability of sufficient data for calibration of morpho-dynamic phenomenon for the river concerned. As such, we can say that this study has following limitations:

- The version of HEC-RAS used in this study cannot divide the sediment load at the junction, so tributaries and distributaries like Mathabhanga, Gorai etc. were not considered in the model
- "Equilibrium Load" was considered at the upstream of the model as enough sediment data was not available to provide for boundary. If more load comes from the upstream, more siltation may occur in the channel
- Rain-fall runoff was not considered.
- Morphological activities cannot fully be interpreted in a one dimensional model. For further understanding the morphological processes in the vicinity of the dredged channel, especially to understand the local phenomenon, a two dimensional model needs to be developed.

ACKNOWLEDGEMENTS

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REFERENCES

FAP24 (1996). "River Survey Project: Hydrology and Morphological Characteristics", Final Report

Hydrologic Engineering Center, US Army Corps (2010). "HEC-RAS River Analysis System: Hydraulic Reference Manual"

IWM (2011). "Mathematical Modelling of the Ganges in connection with the Proposed Ruppur Nuclear Power Plant", Final Report. Institute of Water Modelling, House – 496, Road – 32, New DOHS, Dhaka

IWM (2010). "Impact of Dredging at 3.17km upstream of Hardinge Bridge on the Ganges River", Final Report. Institute of Water Modelling, House – 496, Road – 32, New DOHS, Dhaka

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TWO-DIMENSIONAL SIMULATION OF FLOWS IN BENDS OF AN OPEN CHANNEL BY IRIC NAYS2D

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ABSTRACT

The existences of straight rivers are rare in nature. Most of the natural rivers are meandering type. In a meandering channel, the flow passing through the bend is obviously of a three-dimensional (3D) nature because of the generation of secondary current due to the centrifugal force. Therefore a 3D hydrodynamic model is necessary to accurately simulate the flows in a meandering channel. However, dealing with the practical engineering problems, such as alluvial geomorphic processes, it is not computationally efficient to use 3D models. In such problems, a two-dimensional (2D) model is generally used. In this study, iRIC Nays2D, which is based on 2D mode, is used to simulate the flow field in an open channel with 180^o mildly curved and 180^o sharply curved bends. The flow behavior has been studied at the inlet straight portion, curved portion and outlet straight portion. The simulated velocity distribution and water surface profile were compared with the previous results. For mildly curved channel, simulated results are compared with experimental results and two different Trials. For sharply curve channel, simulated results are compared with experimental results. For The simulated flow fields as well as the water surface profile are found to be well agreed with available previous experiments. Although 180^o bend is an extreme pattern of meandering channel, a successful simulation of such complex flow field by iRIC Nays2D assures us that, it can be applied to any meandering channel.

Keywords: *Open channel flow, 180^o bend, Two dimensional simulation, iRIC, Nays2D, Meandering river.*

1. INTRODUCTION

The existences of straight rivers are rare in nature. Most of the natural rivers are meandering type. At the straight portion of a river, the width-wise and depth-wise variation of flow is very small and only the stream wise velocity or longitudinal velocity dominates the flow. Figure 1 shows a natural river. When flow encounters bend, the water surface increases on outer bank and decreases on inner bank because of the centrifugal force. As a result of continuity of flow and at subcritical state of flow, the velocity decreases near the external wall, and it increases near the internal wall. Changing of water slope, presence of pressure gradient, formation of secondary flow and other characteristics of flow in bends complicate the flow pattern in curved open channel. So far, many researches have been done to explain the stage of flow in bend at open channels and rivers. This subject has more importance when we find out that the complex structure of flow and great turbulence existing in bends causes characteristics such as erosion, deposit, super elevation; score of bed, meander of river and secondary current are arisen. Researchers believe that secondary currents in bends that is created due to eccentric forces and pressure gradient are important factors in changing the above characteristics. Rozovskii (1957) worked on rectangular channels with 180° bend in completely smooth and rough beds. Channel width was 80 cm and bend radius of curvature on the center line was assumed to be 80 cm, so a ratio of R/b = 1 was chosen where, R, was the radius of curvature and b was the width of channel. Discharge value of 12.3 L/sec, flow depth of 5.8 cm, Froude number of 0.35 and Reynolds number of 14000 were selected. He also, performed an extensive research on trapezoidal channels. He illustrated the changes of the tangential velocity distribution at transversal direction and spelt out the cause of generation and intensification of the secondary current through the bend. In these studies no researches were fulfilled on water surface transversal profile at bend entry and bend exit in strongly-curved open channels, also on its varieties through bend. Engelund (1974) described the theory of helical flow in circular bends for a wide rectangular open channel. Leschzm and Rodi (1979) represented a numerical model in conformity with Rozowski tests on a 90° strongly-curved bend with a ratio of average radius to width equal to 1. In that investigation he didn't reach to a linear surface slope at cross section but in both numerical investigation and laboratory model, surface slope near inner bank was obtained greater than external bank. De Vriend (1979) carried out experiments on a 180° bend, which was one of his investigations on a bend at University of Delft, Netherlands. This bend had rectangular cross section with a width of 1.7 m and central radius of 4.25 m. The lengths of straight channel before and after bend were 6 m. Flow discharge for this test was 190 L/sec and flow depth for downstream channel was 0.18 m with Froude number of 0.215.

(a) **(b)**

Figure 1: An example of natural meandering river (a) River of Yamal-tundra in Siberia, (b) Near near Joydevpur-Tangail highway in Bangladesh.

De Vriend and Geoldof (1983) worked on another model with central angle of 90°, discharge of 0.61 m^3/sec, width of 6 m, depth of 0.25 m, 50 m radius of curvature and Froude number of 0.25. In these studies the water surface transversal profile through the bend was linear but water surface elevation was not calculated at upstream and downstream of the bend. Steffler (1985) studied a channel with 270° central angle. This channel had rectangular cross section with width of 1.07 m and depth of 0.2 m. The central radius of curvature is considered to be 3.66 m. Two conditions were considered, one with depth of 6.1 cm and the other with depth of 8.5 cm and bed slope of 0.00083. He presented a new 2D numerical model based on depth-averaged velocity, such that the secondary current having 3D nature was involved and evaluated validity of the model by comparing the numerical and experimental results. After surveying the experimental and numerical results, water surface transversal profile was calculated linear and he didn't comment on it concerning its quantity at bend entry and bend exit. Anwar (1986) worked on 31m length and 180° bends with natural-topography beds. Shimuzu et al. (1990) assumed a logarithmic vertical distribution of the longitudinal velocity and developed a 3D hydrodynamic model. Molls and Chaudhry (1995) developed a 2D depth-average model to solve unsteady flow in open channel. Ye and McCorquadale (1998) worked on two test cases, one was a 270° bend with a one-sided trapezoidal cross section and Froude number of 0.475 and the other was a meander bend consisted of two 90° bends with a width of 2.34 m and central radius of 8.53 m. Length of straight part between two bends was 4.27 m and length of straight approach channels to the bend were 2.13 m. In this research water depth and approach velocity at entry are assumed to be 0.115 m and 0.366 m/sec, respectively. By comparison of 3D hydrodynamic simulation based on k-ε model and investigation of the experimental results, they found out that the secondary current and super-elevation begin upstream the bend and gradually reach the bend. Although, they studied on the water surface longitude profile in inner and outer bank and the axis of bend, they didn't comment on the water surface transversal profile, the beginning and end of super elevation position and effect of transversal distribution of tangential velocities on super-elevation. Blanckaert and De Vriend (2003) also studied a 120° bend, 0.4 m width and central radius of 2 m. However the channel bed was fixed in that research, but using sand with an average grain size of 2.1 mm enabled testing with moveable bed. Bodnar and Prihoda (2006) presented numerical simulation of turbulent free-surface flow accordance with finite-volume method by SST k-ω turbulence model and analyzed strongly-curved bend with 90° angle. In that study, he calculated the nonlinear slope of water surface in the bend. According to investigations of Rozovskii (1957), Leschziner and Rodi (1979), if the ratio of bend radius to the channel width was less than 3, bend was considered strongly curved, otherwise it was mild. In a meandering channel, the flow passing through the bend is obviously of a three-dimensional (3D) nature because of the generation of secondary current due to the centrifugal force. Therefore a 3D hydrodynamic model is necessary to accurately simulate the flows in a meandering channel. However, dealing with the practical engineering problems, such as alluvial geomorphic processes, it is not computationally efficient to use 3D models. In such problems, a two-dimensional (2D) model is generally used. In this study, iRIC Nays2D, which is based on 2D model, is used to simulate the flow field in open channel with 180° mildly and sharply curved bends. The flow behavior has been studied at the inlet straight portion, curved portion and outlet straight portion. The simulated velocity distribution and water surface profile were compared with the previous results.

2. NUMERICAL MODEL AND SIMULATION DETAILS

2.1 iRIC

iRIC (International River Interface Cooperative) is a river flow and riverbed variation analysis software package which combines the functionality of MD_SWMS, developed by the USGS (U.S. Geological Survey) and RIC-Nays, developed by the Foundation of Hokkaido River Disaster Prevention Research Center. The amalgamation of these pieces of software was proposed by Professor Yasuyuki Shimizu (Hokkaido University) and Dr. Jon Nelson (USGS), bringing together the accumulated analysis technology and software developments of MD_SWMS and RIC-Nays. Their vision is to continue to work on cutting-edge technology, incorporating the requests and opinions of our users to develop and provide even more useful software. This software application provides an integrated river simulation environment.

2.2 Nays

Nays2D is an analytical solver for calculation of unsteady 2D plane flow and riverbed deformation using boundary-fitted coordinates within general curvilinear coordinates. The solver's prototype was initially developed by Professor Yasuyuki Shimizu of Hokkaido University in the 1990s; the details of the model are described in Shimizu (2002).

Nays is currently the most sophisticated model within iRIC in terms of handling advection of momentum and strong local unsteadiness. It also includes full sediment-transport capabilities and morphologic change prediction, so the impacts of unsteadiness on bed change can be assessed with this approach. Furthermore, Nays provides a variety of particle-tracking information.

2.3 Basic Flow Equations

Basic equations as obtained by transforming basic equations in an orthogonal coordinates system (x , y) into a general curvilinear coordinate system are shown in Equation (1), (2) and (3):

[Equation of continuity]

$$\frac{\partial}{\partial t}\left(\frac{h}{J}\right) + \frac{\partial}{\partial \xi}\left(\frac{hu^{\xi}}{J}\right) + \frac{\partial}{\partial \eta}\left(\frac{hu^{\eta}}{J}\right) = 0 \tag{1}$$

[Equations of motion]

$$\begin{aligned}
&\frac{\partial u^{\xi}}{\partial t} + u^{\xi}\frac{\partial u^{\xi}}{\partial \xi} + u^{\eta}\frac{\partial u^{\xi}}{\partial \eta} + \alpha_1 u^{\xi}u^{\xi} + \alpha_2 u^{\xi}u^{\eta} + \alpha_3 u^{\eta}u^{\eta} = \\
&\quad -g\left[\left(\xi_x^2 + \xi_y^2\right)\frac{\partial H}{\partial \xi} + \left(\xi_x\eta_x + \xi_y\eta_y\right)\frac{\partial H}{\partial \eta}\right] \\
&\quad -\left(C_f + \frac{1}{2}C_D\alpha_s h\right)\frac{u^{\xi}}{hJ}\sqrt{\left(\eta_y u^{\xi} + \xi_y u^{\eta}\right)^2 + \left(-\eta_x u^{\xi} + \xi_x u^{\eta}\right)^2} + D^{\xi}
\end{aligned} \tag{2}$$

$$\begin{aligned}
&\frac{\partial u^{\eta}}{\partial t} + u^{\xi}\frac{\partial u^{\eta}}{\partial \xi} + u^{\eta}\frac{\partial u^{\eta}}{\partial \eta} + \alpha_4 u^{\xi}u^{\xi} + \alpha_5 u^{\xi}u^{\eta} + \alpha_6 u^{\eta}u^{\eta} = \\
&\quad -g\left[\left(\eta_x\xi_x + \eta_y\xi_y\right)\frac{\partial H}{\partial \xi} + \left(\eta_x^2 + \eta_y^2\right)\frac{\partial H}{\partial \eta}\right] \\
&\quad -\left(C_f + \frac{1}{2}C_D\alpha_s h\right)\frac{u^{\eta}}{hJ}\sqrt{\left(\eta_y u^{\xi} - \xi_y u^{\eta}\right)^2 + \left(-\eta_x u^{\xi} + \xi_x u^{\eta}\right)^2} + D^{\eta}
\end{aligned} \tag{3}$$

where,

$$\alpha_1 = \xi_x\frac{\partial^2 x}{\partial \xi^2} + \xi_y\frac{\partial^2 y}{\partial \xi^2}, \alpha_2 = 2\left(\xi_x\frac{\partial^2 x}{\partial \xi\partial \eta} + \xi_y\frac{\partial^2 y}{\partial \xi\partial \eta}\right), \alpha_3 = \xi_x\frac{\partial^2 x}{\partial \eta^2} + \xi_y\frac{\partial^2 y}{\partial \eta^2}$$

$$\alpha_4 = \eta_x\frac{\partial^2 x}{\partial \xi^2} + \eta_y\frac{\partial^2 y}{\partial \xi^2}, \alpha_5 = 2\left(\eta_x\frac{\partial^2 x}{\partial \xi\partial \eta} + \xi_y\frac{\partial^2 y}{\partial \xi\partial \eta}\right), \alpha_6 = \eta_x\frac{\partial^2 x}{\partial \eta^2} + \eta_y\frac{\partial^2 y}{\partial \eta^2}$$

$$D^{\xi} = \left(\xi_x \frac{\partial}{\partial \xi} + \eta_x \frac{\partial}{\partial \eta}\right)\left[v_t\left(\xi_x \frac{\partial u^{\xi}}{\partial \xi} + \eta_x \frac{\partial u^{\xi}}{\partial \eta}\right)\right] + \left(\xi_y \frac{\partial}{\partial \xi} + \eta_y \frac{\partial}{\partial \eta}\right)\left[v_t\left(\xi_y \frac{\partial u^{\xi}}{\partial \xi} + \eta_y \frac{\partial u^{\xi}}{\partial \eta}\right)\right]$$

$$D^{\eta} = \left(\xi_x \frac{\partial}{\partial \xi} + \eta_x \frac{\partial}{\partial \eta}\right)\left[v_t\left(\xi_x \frac{\partial u^{\eta}}{\partial \xi} + \eta_x \frac{\partial u^{\eta}}{\partial \eta}\right)\right] + \left(\xi_y \frac{\partial}{\partial \xi} + \eta_y \frac{\partial}{\partial \eta}\right)\left[v_t\left(\xi_y \frac{\partial u^{\eta}}{\partial \xi} + \eta_y \frac{\partial u^{\eta}}{\partial \eta}\right)\right]$$

$$\xi_x = \frac{\partial \xi}{\partial x}, \xi_y = \frac{\partial \xi}{\partial y}, \eta_x = \frac{\partial \eta}{\partial x}, \eta_y = \frac{\partial \eta}{\partial y}$$

$$u^{\xi} = \xi_x u + \xi_y v, u^{\eta} = \eta_x u + \eta_y v$$

$$J = \frac{1}{x_{\xi} y_{\eta} - x_{\eta} y_{\xi}}$$

2.4 Flow Parameters

The hydraulic parameters for different test cases that were studied are shown in Table 1. The parameters in case I are taken as same as the experiments by De Vriend (1979). The parameters in case II are taken as same as the experiments by Rozovskii (1961).

Table 1 Hydraulic parameters for the simulation of simulated cases

Case no.	L (m)		W, m	Q_o, m^3/s	h_o, m	Bottom slope, S_o	Δt, sec	R/B	n	Courant number
	U/S	D/S								
I	6	6	1.7	0.0671	0.1953	0.016	0.020	3.5	0.0350	0.0050
II	6	6	0.8	0.0123	0.0600	0	0.001	1.0	0.0102	0.0143

Here, Δt = Time step, L = Length of the straight portion, Q_o = Upstream discharge, h_o = Mean depth, S_o = Bottom slope, n = Manning's coefficient.

2.5 Computational Schemes and Flow Domain

The governing equations for mean velocities and turbulent flows are discretized with the finite difference method based on full staggered boundary fitted coordinate system. The basic equations are discretized as fully explicit forms and solved successively with the time increment in step by step. It is solved using iterative procedure at each time step. Constant discharge at upstream and constant depth with zero velocity gradients was given as downstream boundary conditions. Finite volume method was used for the solution of the equations. 2D grid, Q, h_e and initial flow conditions were given as input. Figure 2 shows the sketch of the flow domain for Case I which is performed under the same conditions of the experiments conducted by De Vriend (1979) and Figure 3 shows the sketch of the flow domain for Case II which is performed under the same conditions of the experiments conducted by Rozovskii (1961).

2.6 Preparation of Grids for Numerical Simulation

The grids are prepared for the simulation and grids are given in Figure 4. The computational mesh consists 126x25 grids, where 126 grids in the longitudinal direction and 25 grids in each of the cross sections for Case I (Mildly curved channel). For Case II (Sharply curved channel), the computational mesh consists 158x23 grids, where 158 grids in the longitudinal direction and 23 grids in each of the cross sections.

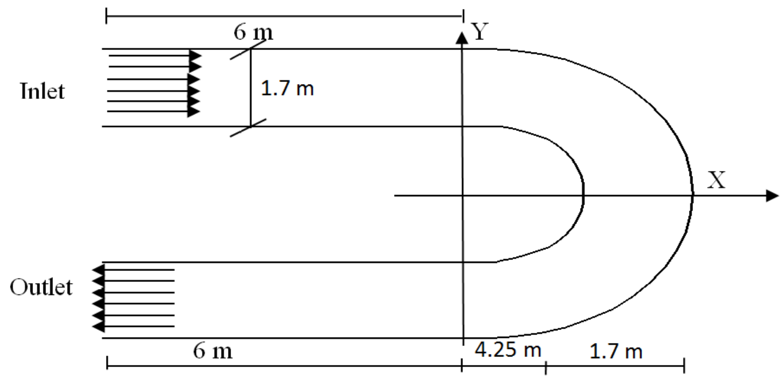

Figure 2: Definition sketch of an open channel with 180° mildly curved bend for case I

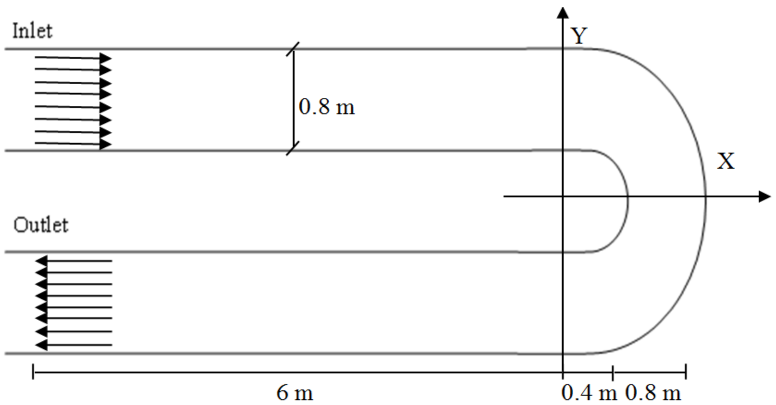

Figure 3: Definition sketch of an open channel with 180^0 sharply curved bend for case II

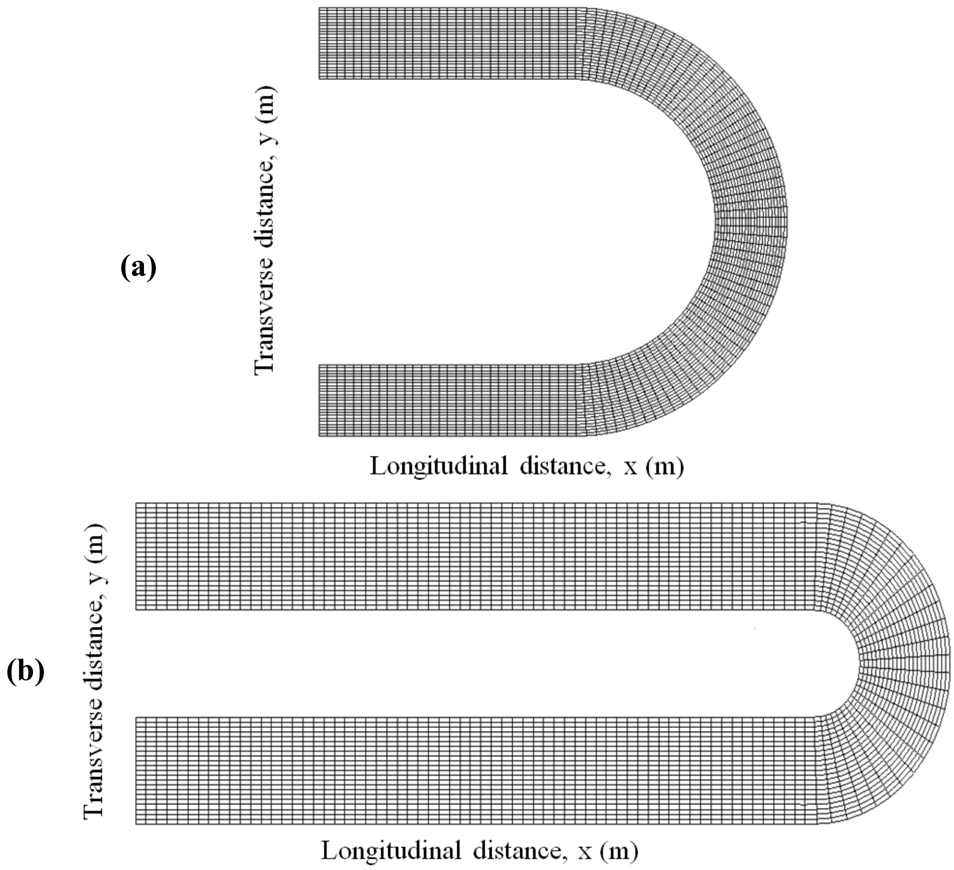

Figure 4: Input grid for simulation (a) Case I, (b) Case II

3. RESULTS AND DISCUSSION

3.1 Simulated Result Of Case I (Mildly Curved Channel)

Among the two cases, for the Case I, de Vriend (1979) experiment, the channel consists of 1.7 m wide flume having a U-shaped ground plan. The bottom of the channel is horizontal and the sidewalls are vertical. The radius of curvature of the flume axis in the bend is 4.25 m and the upstream and downstream straight reaches have an effective length of about 6.0 m. The ratio of radius of curvature to channel width is 3.5. The discharge at the inlet is 0.0671 m^3/s, the averaged velocity is 0.202 m/s and the averaged flow depth is 0.1953 m. The logarithmic laws at the sidewalls were used in the numerical simulation. The simulation reach consists of

the U bend and the effective straight reaches at the upstream and downstream. Constant discharge at upstream and constant depth with zero velocity gradients was given as downstream boundary conditions.
Mildly curved channel (case I) is simulated for two trails.

- Trail 1: It consists of Standard Solver type with finite differential method and discretizing the advection terms using CIP method.

- Trail 2: It consists of Advanced Solver type with finite differential method and discretizing the advection terms using CIP method with turbulent model as K-epsilon model.

Figure 5 plots the simulated flow field as velocity vector and the surface elevation in shaded color. As there is no significant difference in the results between Trial 1 and Trial 2, in Figure 5 (b) the results are presented for Trial 1 which represents that velocity vectors at inlet and outlet are uniform. At the bend the velocity along the inner bank is found higher than the outer bank. The velocity vector is found to be comparable with experimental results as shown in Figure 5(a).

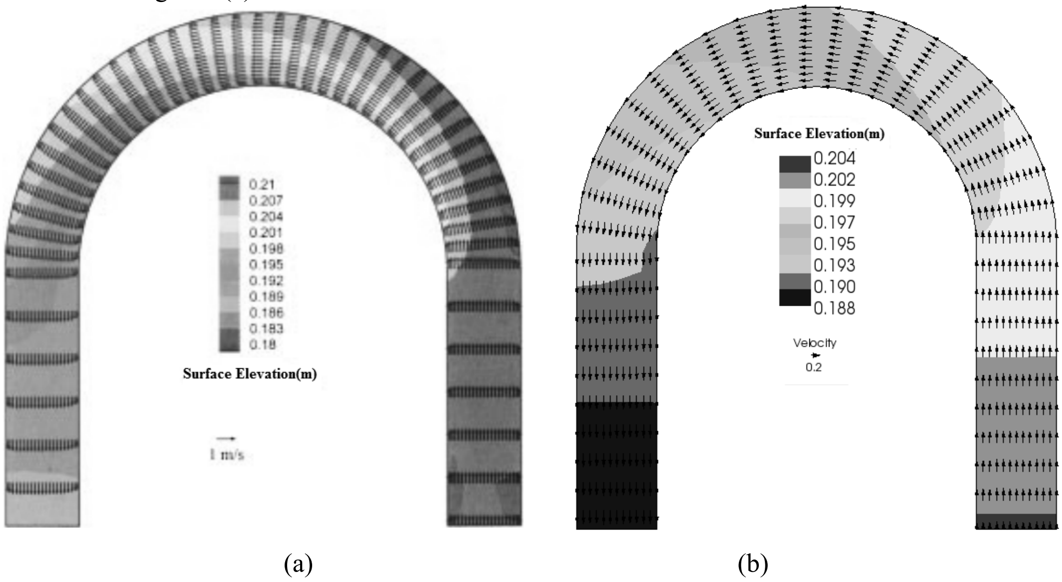

Figure 5: Velocity vector super imposed on the surface elevation contour for Case I, (a) Experiment (De Vriend, 1979) (b) Simulation

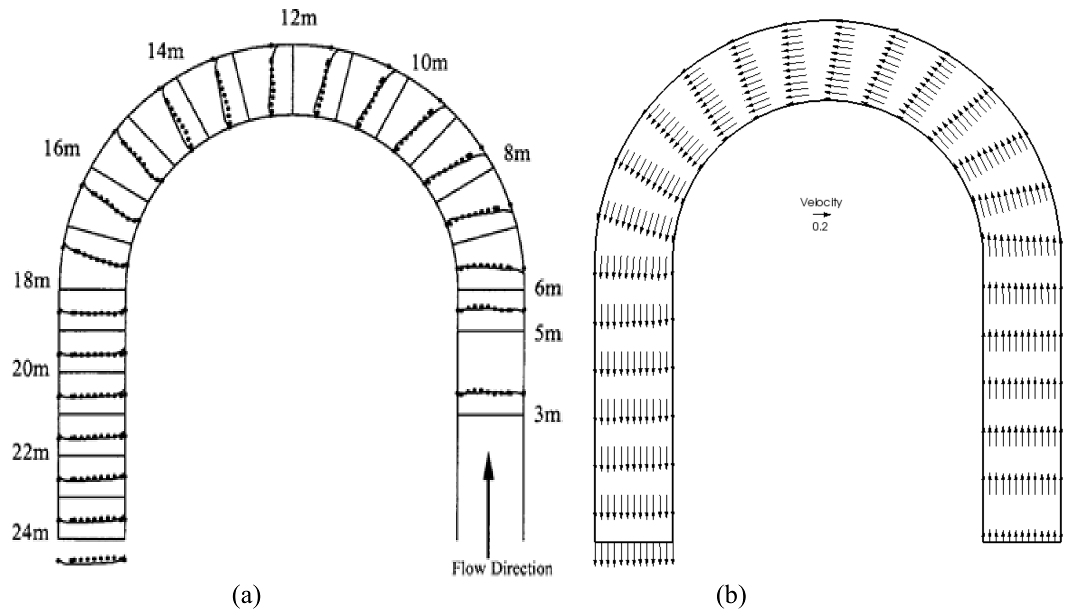

Figure 6: Distribution of depth-averaged velocity for Case I, (a) Experiment (De Vriend, 1979), (b) Simulation

The comparison of the simulated velocity with previous experiments is shown in Figure 6. Figure 6 (a) and (b) shows the experimental velocity distribution and simulated velocity distribution, respectively, where Figure 6 (b) is for Trial 1. It is observed that the velocity along the inner bank is longer than the outer bank. The acceleration of the stream wise velocity at the inner bank is due to the transverse convection of momentum induced by the secondary current. However, for this mild bend, the radius of channel curvature is much larger than the width of the channel and thus the effect of secondary flow not very strong.

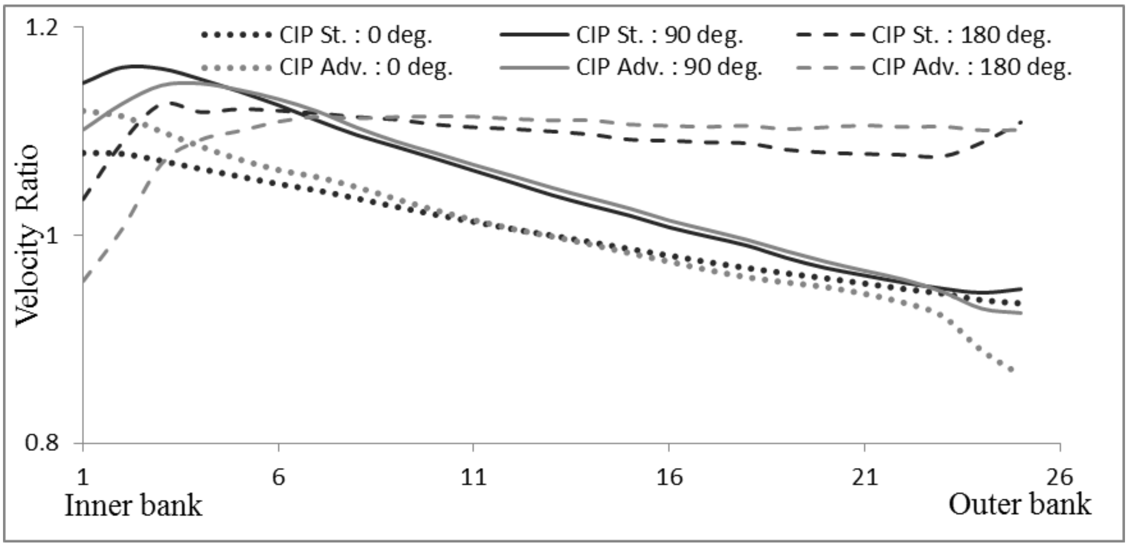

Figure 7: Comparison of simulated velocity ratio at different sections for trial 1 and 2 by for Case I

Figure 7 plots the variation of ratios of the simulated velocity with the mean velocity along the width. Variation of results between Trial 1 and Trial 2 is exposed in Figure 3.3. The velocity ratio is expressed as, $V_r = \frac{\sqrt{(u^2+v^2)}}{u_0}$; where; u = longitudinal velocity, v = transverse velocity, and u_0 = mean velocity. The mean velocity was taken as 20 cm/s. From the Figure 7 it is clear that, the velocity ratio is higher at the inner bank and gradually decreases towards the outer banks. This trend is similar at the entrance of the bend (at θ=0), middle of the bend (at θ=90^0) and exit of the bend (at θ=180^0). The magnitude of this ratio at 180^0 is higher than 90^0 and 0^0 at outer bank and lower in inner bank of channel. For 90^0 and 180^0, higher velocity ratio is found in inner bank for Trial 1 than Trial 2 which is reverse along the transverse direction toward the outer bank, except outer bank. For 0^0, higher velocity ratio is found in inner bank for Trial 2 than Trial 1 which is reverse in outer bank that is changing gradually.

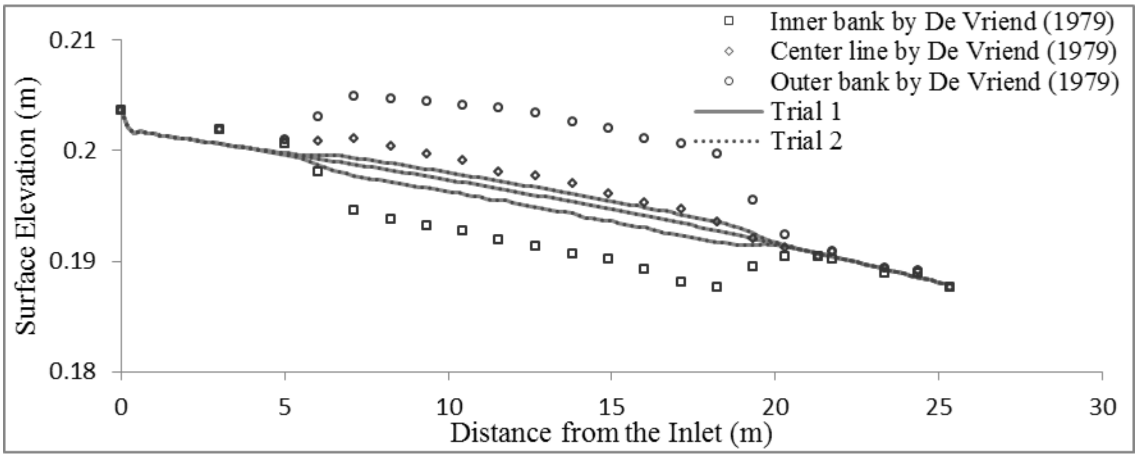

Figure 8: Comparison of simulated water surface profile for Trial 1 and 2 with previous experimental results for Case I

The comparison of simulated water surface elevation with the experimental results at the inner bank, centerline, and outer bank was plotted in Figure 8. It shows that the water surface elevation at the outer bank is much higher and at the inner bank is much lower than that of center of the channel water surface elevation throughout the bend. The rise of flow at the outer bank results from the centrifugal force. This change in water surface elevation starts from just before to the entrance of the bend and continued up to just after the exit of the bend Although the velocity field agreed well with the experiment, the fluctuation of water surface in the bend is found lower than the experimental results. However, the present simulation is found to be well compared with the simulation by Ahmadi et al. (2009). It is observed that the difference of results of Trial 1 and Trial 2 is negligible. The simulated result is shown good comparison with experiment.

3.2 Simulated Results of Case II (Sharply Curved Channel)

For case II, Rozovskii's (1961), the channel consists of 0.8 m wide flume having a U-shaped ground plan. The straight reaches at the upstream and downstream have an effective length of about 6.0 m. The internal and external radius of curvature is 0.4 m and 1.2 m respectively. The ratio of radius of curvature to channel width is 1.0. The cross section of the channel bend is a 0.8 m wide rectangle, and entire channel bend is horizontal. The channel bottom is made smooth. The discharge at the inlet is 0.0123 m^3/s, the averaged mean velocity is 0.265 m/ s, and the averaged flow depth is 0.06 m. The computational mesh is 158 x 23 with 158 cross sections in the longitudinal direction and 23 nodes in each cross section. The logarithmic law at the sidewalls was used in the numerical simulation. The simulation reach consists of the U bend and the effective straight reaches at the upstream and downstream. Constant discharge at upstream and constant depth with zero velocity gradients was given as downstream boundary conditions. Sharply curved channel (case II) is simulated of Standard Solver type with finite differential method and discretizing the advection terms as upwind scheme by iRIC Nays. iRIC Nays results are compared with experiment.

Figure 9 (a) and (b) shows the experimental and simulated by iRIC Nays contour plots of water surface, respectively. It can be seen that the numerical model successively reproduces well the higher water surface elevation along the outer bank than the inner bank. The water surface elevation is also higher at the inlet straight portion than the outlet straight portion. So the overall agreement between measured and simulated profiles appears to be quite good.

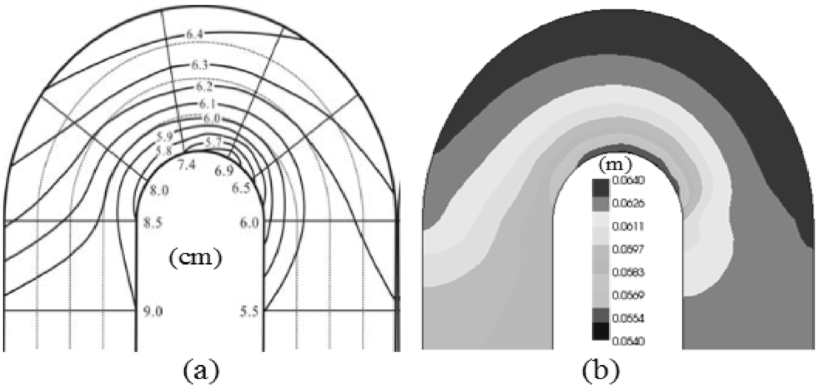

Figure 9: Distribution of water surface elevation for Case II, (a) Experiment (Rozovskii, 1961), (b) Simulation

The longitudinal distribution of water surface along the inner bank and outer bank for experiment and simulation are plotted in Figure 10. The horizontal axis in the figure is the dimensionless distance from the entry of the channel bend. From this figure it is clear that, the water surface elevation at the outer bank is much higher than that in the inner bank throughout the bend. It can be also seen that the computed flow depth is close to the measured data. So the overall agreement between measured and simulated profiles appears to be quite good.

Figure 11 plots the variation of ratios of the simulated velocity with the mean velocity along the width. The velocity ratio is expressed as, $V_r = \frac{\sqrt{(u^2+v^2)}}{u_0}$; where; u = longitudinal velocity, v = transverse velocity, and u_0 = mean velocity. The mean velocity was taken as 26.5 cm/s. From the Figure 11, it is clear that the velocity ratio is higher at the inner bank and gradually decreases towards the outer banks. This trend is similar at the entrance of the bend (at θ=0) and middle of the bend (at θ=90^0). But exit of the bend (at θ=180^0) trend is reverse order along a small portion of width and decrease the ration slightly again.

Figure 12 (a) and (b) presents the measured (Rozovskii, 1961) and simulated velocity vector, respectively. The simulated result by iRIC Nays is found to be good comparison with the experiment. Figure 4 (b) represents that the velocity is uniform at inlet, higher in inner bank at bend and higher in outer bank at outlet portion. The overall agreement between measured and simulated flow field appears to be quite good.

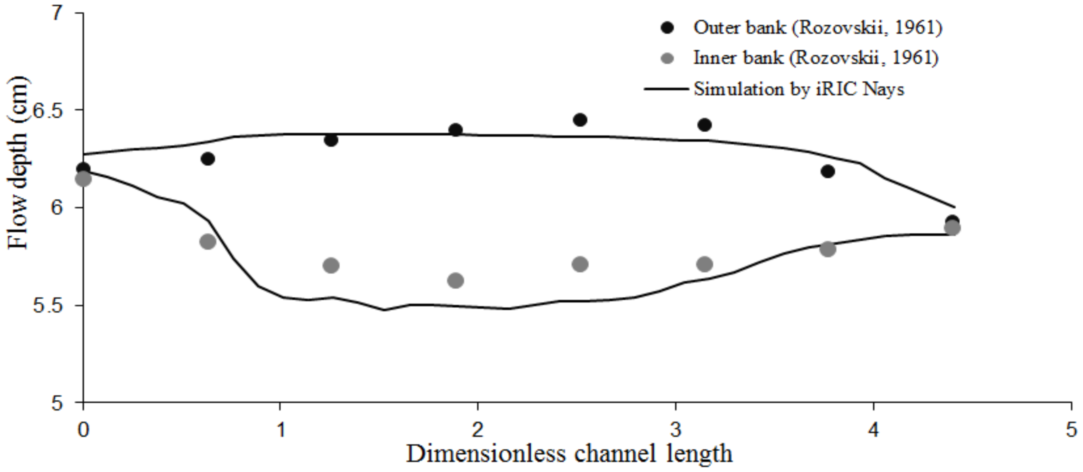

Figure 10: Comparison of simulated water surface profile with experimental results for Case II

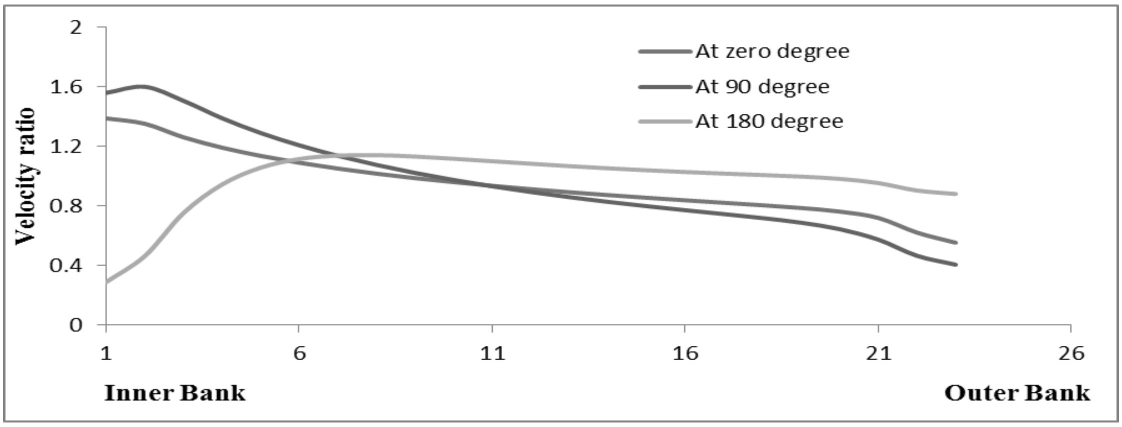

Figure 11: Comparison of simulated velocity ratio at different sections for Case II

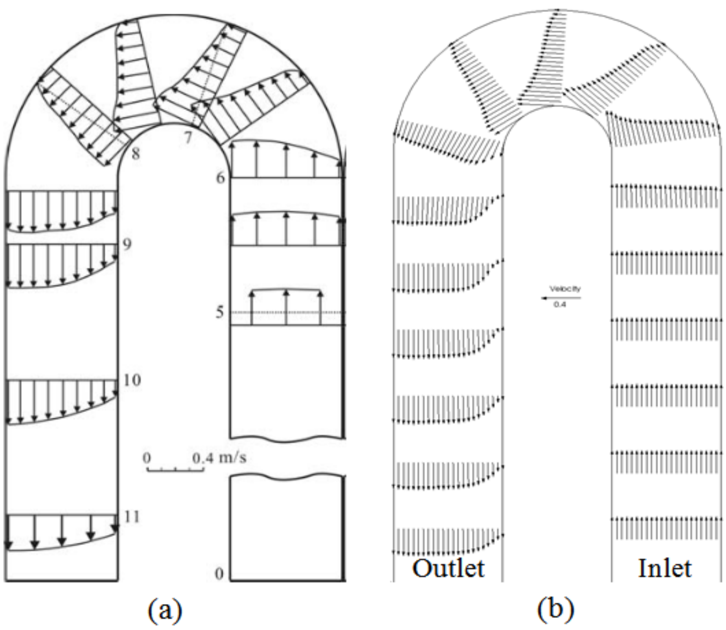

Figure 12: Comparison of width wise variation of velocity vector at different section of the channel for Case II, (a) Experiment (Rozovskii, 1961), (b) Simulation

4. CONCLUSION AND RECOMMENDATION

4.1 Summary of the Findings

In this study, flows in mildly and sharply curved channel with 180^o bend have been simulated by using iRIC Nays2D.

For both the Cases, the velocity vector at inlet and outlet straight portion is found uniform but at the bend, velocity vector along the inner bank is found higher than the outer bank. The water surface elevation at outlet is observed higher than the inlet straight portion. The water surface elevation at the outer bank is found much higher and at the inner bank is found much lower than that of center of the channel. The simulated velocity vector and water surface profile for case I is compared with experimental results. For case II, the simulated velocity vector and water surface profile is also compared with previous experimental results. The simulated result is found to be well agreed with the experimental result. For a mildly curved channel, the radius of curvature of the bend is much higher than the width of the channel but for a sharply curved channel, the radius of curvature is in the order of the width of the channel at the bend. So the effect of secondary flow is much strong for a sharp bend than the mild bend. Therefore the ratio of velocity is much higher for sharp bend compared to mild bend.

4.2 Conclusions

It can be concluded that, for the sharply and mildly curved channel, the simulation predicts the flow field accurately at the inlet straight portion, bend as well as at the outlet straight portion of the channel. For the sharply curved channel, the simulation predicts the flow field of the channel more accurately than mildly curved channel. So, iRIC Nays2D is suitable to deal a channel flow field which is highly affected by secondary current. The simulated flow field and water surface profile for 180^o bends obtained from the two-dimensional model showed good comparison with experiments. Although the fluctuation of in depth is not well represented by iRIC Nays2D, the global flow feature and velocity profile are found to be well agreed to the experiment. Therefore, it can be conducted that iRIC Nays2D is applicable for simulating the flow field in bends and hence for the meandering channels.

4.3 Recommendation for Further Study

In this study, the flow fields in a sharply and mildly curved channel have been studied using iRIC Nays2D. Since the flow in a bend is highly three-dimensional, it is recommended that for detailed study of the flow field, a 3D numerical model needs to be employed.

The study can be extended to simulate the sediment and nutrient transport in a bend or meandering channel using the same iRIC Nays2D software.

REFERENCES

Ahmadi, M. M., Ayyoubzadeh, S. A., Namin, M. M., and Samani, J. M. V. (2009). “A 2D depth-averaged model for simulating and examining unsteady flow patterns in open channel bends”. Department of Water Structures Engineering, College of Agriculture, Tarbiat Modares University, P. O. Box: 14115-336, Tehran, Islamic Republic of Iran.

Callander, R. A. (1978). “River meandering”. Ann. Rev. Fluid Mech. 10,129–158.

Chow, V.T. (1959). “Open-channel hydraulics”. Mc Graw-Hill Book co., Inc., New York, N.Y.

De-Vriend, H.(1979). “Flow measurements in a curved rectangular channel.” Internal. Rep. No. 9-97. Tech. Rep., Laboratory of Fluid Mechanics, Delft Univ. of Technology, Delft, The Netherlands.

De-Vriend, H. (1980). “Velocity redistribution in curved rectangular channels.” J. Fluid Mech., 107, 423–439.

De-Vriend, H. (1981). “Flow measurements in a curved rectangular channel. Part 2: Rough bottom.” Internal Rep. No. 5-81. Tech. Rep., Laboratory of Fluid Mechanics, Dept. of Civil Engineering, Delft Univ. of Technology, Delft, The Netherlands.

Jennifer G. Duan (2004). “Simulation of flow and mass dispersion in meandering channel.” Journal of Hydraulic Engineering. ASCE/ October 2004, pp.964-976

Kim, T.B. and Choi, S.U. (2003). “conducted an experiment on numerical simulations of open-channel flows in a bend using the finite element method”. School of Civil and Environmental Engineering, Yonsei University, Seoul, Korea.

Mahjabin, T. (2011). “Behavior of open channel flows with a rectangular side cavity”. Khulna University of Engineering & Technology (KUET).

Mondol, M.S. (2012). “Two-dimensional simulation of flows in bends of an open channel”, Undergraduate Thesis, Khulna Univ. of Engg. & Tech, Bangladesh.

Nezu, I., and Nakagawa, H. (1993). “Turbulence in open channel flows.” IAHR Monograph, Rotterdam, The Netherlands, 53-56.

Rozovskii, I.L. (1961). “Flow of water in bends of open channels.” Academy of Science of krainian SSR, Russia.

Shimizu, Y. (2002). A method for simultaneous computation of bed and bank deformation of a river, Proceedings of the International Conference on Fluvial Hydraulics, River Flow 2002, Louvain-la-Neuve, Belgium, vol. 2, pp.793-802, September, 2002.

Tan, W. (1992). “Shallow water hydraulics.” Elsevier Oceanography Series, 55. Elsevier Science Publishers BV, Amsterdam, The Netherlands.

GEOSTATISTICAL ANALYSIS OF GROUNDWATER LEVEL FLUCTUATIONS IN THE SHALLOW AQUIFER OF NORTHWESTERN BANGLADESH

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ABSTRACT

Management of groundwater resources plays a vital role in conserving the sustainable conditions of an aquifer. Modern tools and techniques that can reveal the critical conditions of water resources are thus indispensable for the effective groundwater management. In this paper, application of a well-known geostatistical technique, kriging, for the spatial analysis of groundwater level fluctuations is explored. Rajshahi district in the northwestern Bangladesh is taken as a case study area, which has experienced considerable variations in groundwater levels due to intensive exploitation of groundwater for increased irrigation expansion. Groundwater level records obtained from 25 monitoring wells for 13 years (1996-2008) are collected from the Bangladesh Water Development Board (BWDB). Each year is divided into two seasons (wet and dry) and total 26 empirical variograms are calculated for each set of data. Exponential, Gaussian and Spherical variogram models are fitted to the empirical variograms and total 78 variogram models are obtained. For each set of data, a best variogram model is chosen by rigorous trial and error wherein 4 goodness-of-fit measures (RMSE, MAE, R, RSS) are used for model identification and parameters optimization. The finally selected models are then used by ordinary kriging in ArcGIS platform to estimate the groundwater levels and estimation variance (which express the accuracy of the estimated groundwater levels). Results demonstrate that a strong spatial structure existed for groundwater level fluctuations due to very low nugget effects. On average, the range of variograms for spatial analysis is found as 25.5 km in wet season and 42.3 km in dry season, respectively. Ordinary kriging technique with cross-validation is applied to assess the accuracy of the estimation of the groundwater level fluctuations for spatial scales. Cross-validation results indicate the appropriateness of chosen variogram for kriging analysis. Results of ordinary kriging revealed that groundwater level fluctuations were underestimated by only 5% for spatial analysis, which are acceptable errors and support the unbiasedness hypothesis of kriging. Groundwater levels were also interpolated by generally used Inverse Distance Weighting (IDW) method and it is found that IDW method exhibits higher errors as compared to ordinary kriging method. This study, therefore, proves that kriging generates spatial maps of groundwater level fluctuations with low uncertainty and can be used as an appropriate tool for identifying the critical regions where more attentions for sustainable use of groundwater are necessary. The study finally suggests that in the identified critical areas water harvesting systems should be practiced to recharge the groundwater, and current cropping pattern should be altered to another one that need less water requirement.

Keywords: *Geostatistics, Groundwater levels, Ordinary Kriging, Semivariogram, Shallow aquifer, Northwestern Bangladesh*

1. INTRODUCTION

Northern part of Bangladesh is now facing water scarcity problems in both agriculture and secured livelihood (Alice Mbugua, 2011). The Barind area in Rajshahi zone of Bangladesh greatly depends on ground water for irrigation and other municipal water requirement (Ahmeduzzaman et al., 2012). With the mounting use of ground water for cultivation, municipal, industrial needs, the annual extraction of groundwater is going far from the net average recharge from natural sources. As a result groundwater is being withdrawn from storage without enough recharge and water levels are lowering resulting in crop failure. Thus for the last few decades Bangladesh is experiencing water related difficulties like river bed Siltation, low water flow, scarcity of water for irrigation etc. This northern part is 37 meter above sea level. People of this region used to cultivate rice once a year, but at present they are producing various crops round the year. As a result, they are facing water scarcity problem. Most of the crops need more irrigation water for production. For high yield of crops water is therefore a vital factor. To investigate the problem associated with groundwater in this region, Barind Multipurpose Development Authority (BMDA) did a survey. They reported on their 71st board meeting that, 7.52 lakh hectares of farming fields along with 2.61 lakh hectares of irri-boro land through operating 14,090 power-driven deep tubewells (DTWs) almost everywhere in the country's northwest zone. These irrigated lands have yielded almost 61.08 lakh tonnes of extra crops especially food grain in the region especially the vast Barind Tract yearly. To have proper monitoring on groundwater level and agriculture practice smooth, analysis work must be done. For better crop yield it is a must to keep the water table within suction limit. BMDA (Barind Multi-purpose Development Authority) have taken necessary protective measures to ensure the annual withdrawal less than the annual recharge to keep the GW level in position. This is an optimistic measure for surely. They estimated GW recharge in the area at least one-third of the annual rainfall and that was about 500 mm/year (Asaduzzaman and Rushton, 2006). Islam and Kanumgoe (2005) estimated the long-term annual average recharge of 152.7 mm using water balance study and aquifer simulation modeling. A government report suggests that recharge to GW in the northwestern part varies from 210 to 445 mm. However, exploitation of GW in the area is going on the basis of one-third rainfall recharge hypothesis of BMDP, which is beyond the sustainable yield according to Islam and Kanumgoe (2005). Therefore, study related to GW depletion is an vital requirement in the northwestern region of Bangladesh.

2. METHODOLOGY

Methodology of this study begins with the selection of study area. The groundwater level data is collected from BWDB. Then the data is processed and divided into dry and wet periods. The data is checked for normal distribution with histogram tool, QQ plot, trend analysis and semivariogram cloud. Geostatistical analysis is done under kriging method using ArcGIS platform. Three types of variogram are generated with the groudwater level data. The best variogram model is chosen based on different types of prediction errors. Finally, the corresponding groundwater surface maps are generated.

2.1 Study Area

Rajshahi district, under northwestern region of Bangladesh is selected as the study area. The location of study area in Bangladesh and location of observation well stations used for the collection GWL data are shown in Figure 1.

2.2 Data Collection

After selecting the study area (Rajshahi district in Bangladesh), GW level data are collected from the Groundwater Circle (GWC) of Bangladesh Water Development Board (BWDB), Dhaka for 25 GW monitoring stations. Location of data collection points are shown in Figure 1. After processing and analysis the GW time series data, the cumulative deficit is calculated. Long-term weekly GW level fluctuations data are available from; 1996-2008. Certain kriging method works best if the data is approximately normally distributed. To run this analysis work on Geostatistical analysis tool the distribution of data must be checked. This distribution of data is checked for normality by the following criteria. The details of the collected data are presented in table 1.

2.2.1 Histogram Tool

All the collected data is checked with histogram tool. The groundwater level data for dry season 1996 is checked by histogram tool. The mean and median value differs from each other. By Log transformation mean and median values become very close. This is the confirmation of the distribution of input data. All the data for other years are checked with histogram tool. Histogram analysis of GWL data for dry period in the year 1996 is shown in Figure 3 and 4.

2.2.2 QQPlot

The QQPlot is another useful tool to check the normality of the distribution of data. Data of dry period of 1996 is plotted with QQ plot. Some points are scattered from the straight line. By log transformation this data came close to the straight line. Similar work is does for all other years' data. QQPlot of GWL data for dry period in the year 1996 is shown in Figure 5 and 6.

2.2.3 Trend Analysis

Before kriging trend (if present in dataset) has been removed. Global trends of the input dataset are identified by the trend analysis tool. The data for dry season of 1996 is plotted on trend analysis tool. By applying 30˚ rotation, it is found that there is U shaped curve present. This indicated that second degree trend is present on the data. This trend is removed for generating a smooth surface in kriging method**.** Trend analysis of GWL data for dry period in the year 1996 is shown in Figure 7.

2.2.4 Semivariogram Cloud

To examine spatial correlation between the measured sample points semivariogram clouds are used. The semivariogram cloud value for dry period 0f 1996 is obtained by the difference squared between values of each pair of location, plotting on y axes and the distance separating each pair plotting on x axes. From this semivariogram cloud global and local outliers are identified. The data set shows the local and global outliers present on the data. Using the semivariogram cloud outliers are selected for other years' data. Semivariogram cloud of GWL data for dry period in the year 1996 is shown in Figure 8.

2.2.5 Conversion of location coordinates

Map projection on GIS follows the method of representing the three dimensional earth surface into a two dimensional plane. For the accuracy of projection the conversion of coordinates have been done. Universal Mercator Projection (UTM) is a widely used projection system. Now a day this system is used for generating digital maps. In this system the earth (between 80˚S and 84˚ N latitude) is divided into sixty zones, each a six degree band of longitude and a secant transverse Mercator is used in each zone. The longitude of Bangladesh lies between 88˚E and 92˚E. The UTM system uses 6˚ band on longitude to divide the earth into zones. Bangladesh has fallen into two Different UTM zones. For this reason problem arises while mapping a zone of Bangladesh. To minimize this problem Flood Action Plan 19 (FAP 19) studied a new projection system known as BTM projection system. This BTM system defines some necessary parameters. For mapping on ArcGIS these parameters can be used to set BTM projection.

2.2.6 Kriging Method

The kriging is started with the selection of the proper attribute. The trend is removed by before plotting different variogram model. Calculation is done to determine the values of nugget, lag size, lag numbers, range and partial sill. To achieve the best fitted variogram these parameters are used. Five neighbors are included and sector type is used as ellipse. Angle is used as 45 for better cross- validation. All the error data are noted and the measured and predicted data was also taken onto account for the generation of surface map. Exponential, gaussian and spherical models and their corresponded surface maps were generated. Comparing the Mean Error (ME), Root Mean Square Error (RMSE), Average Standard Error (ASE), Mean Standardized (MS), Root Mean Square Standardized (RMSS) of these models, the best variogram is chosen. To provide the best prediction the model must satisfy the following criteria

- The Mean Error (ME) should be zero.
- Root Mean Square Error (RMSE) and Average Standard Error (ASE) should be as small as possible.
- Root Mean Square Standardized (RMSS) should be close to 1.

2.2.7 Generation of Different Variograms

Different types of variograms are generated with calculated parameters such as nugget, partial sill, range, lag size and number of lags. In this study exponential, Gaussian and spherical variograms are generated for producing surface maps. Different variograms for dry period in 1996 is shown in Figure 9. The regression equations for different variograms are as following:

Regression equation for Spherical model = 0.754 x + 3.051
Regression equation for Gaussian model= 0.754 x + 3.053
Regression equation for Exponential model= 0.630 x + 4.500

2.2.8 Cross Validation

Cross validation indicates the acceptance of the model in terms of predict the unknown values. Taking on account all points cross-validation omits one point, then measures its value with respect to all other values and compares the measured and predicted values. The main objective of cross-validation is to make the informative decision of choosing the best model that gives accurate predictions. The model which gives best prediction is obtained by evaluating the prediction errors. Figure 10 shows the cross-validation for dry period in 1996.

2.2.9 Generation of Surface Maps

Figure 11 shows surface maps for exponential, Gaussian and spherical variogram models for both dry 1996. The maps show the variation of GW level at different locations of Rajshahi district. From cross-validation error data Gaussian model is chosen as best model. Gaussian and spherical models were very close by the prediction errors. The surface maps also show that GW level fluctuation is almost same for these two models. In the middle portion of the map, the GW level comprises within 8-11 mPWD. This zone is identified as critical zone and facing scarcity of GW.

3. ILLUSTRATIONS

3.1 Figures and Graphs

The location of study area in bangladesh and data collection points is shown in figure 1.

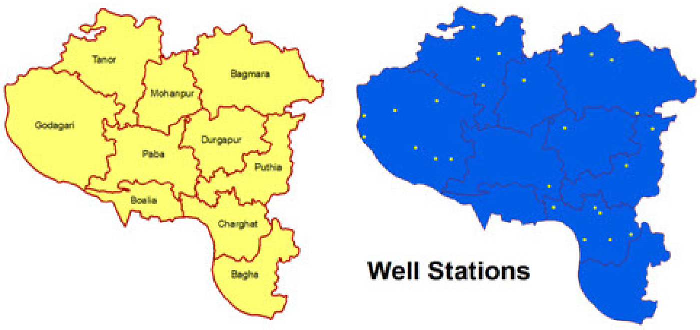

Figure 3: Study area and data collection points

GW level data is available in 25 stations in the study area. These data is used to plot GW hydrograph with 12-month moving average. The pattern of groundwater level fluctuation is observed by plotting hydrograph. In this study 13 years' (1996-2008) GW level data is analysed. The analysis shows 7 types of nature of hydrograph for all 25 monitoring stations. Figure 2 shows two different types of hydrograph at two diffent stations for groundwater level data from 1996 to 2008.

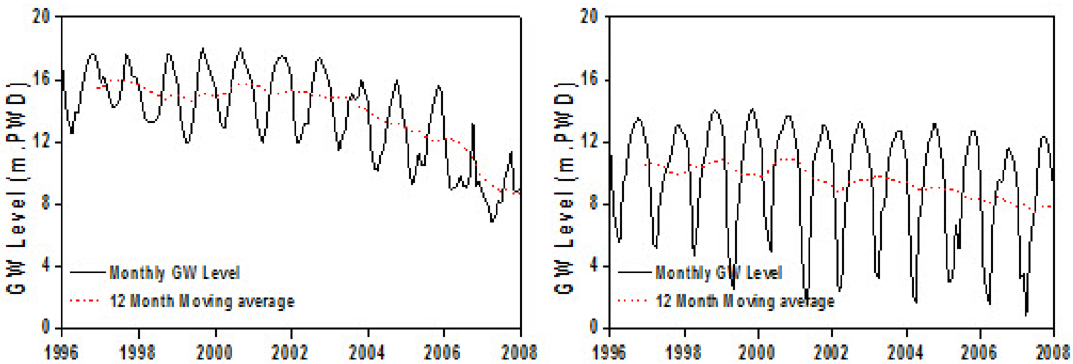

Figure 4: Hydrographs for groundwater level at two monitoring stations.

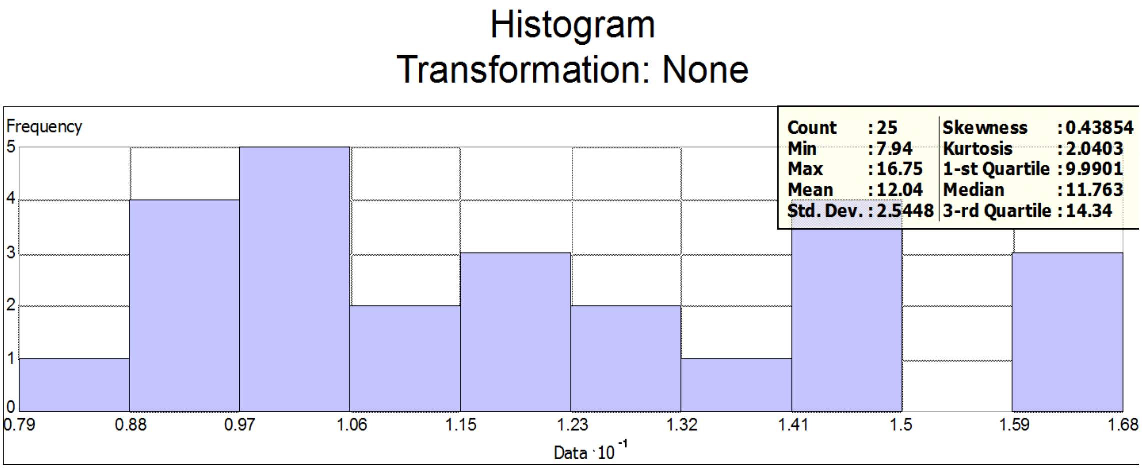

Figure 5: Histogram plot of GWL data (dry) in 1996

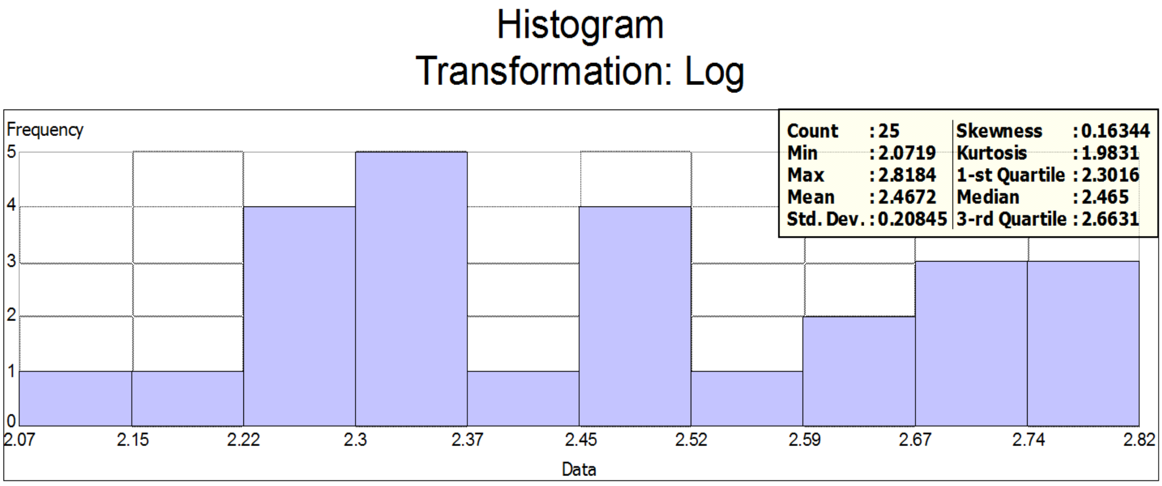

Figure 6: Histogram plot(log) of GWL data (dry) in 1996

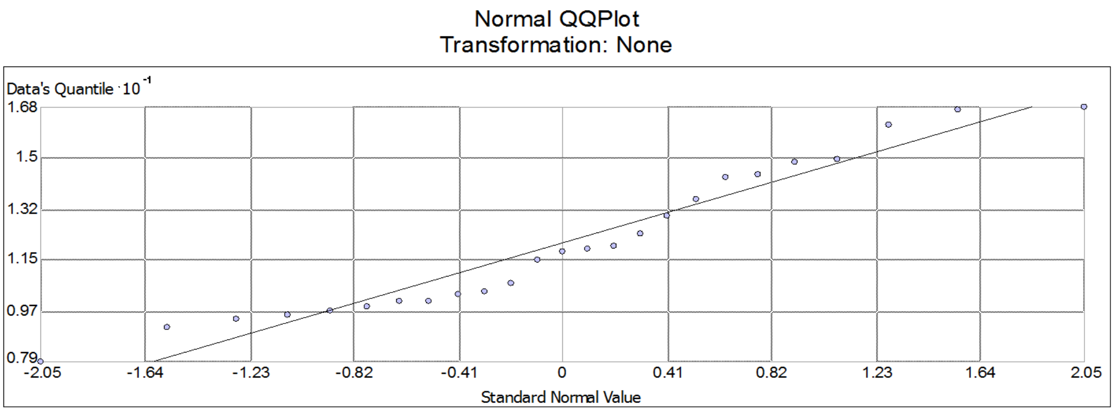

Figure 7: QQPlot of GWL data (dry) in 1996

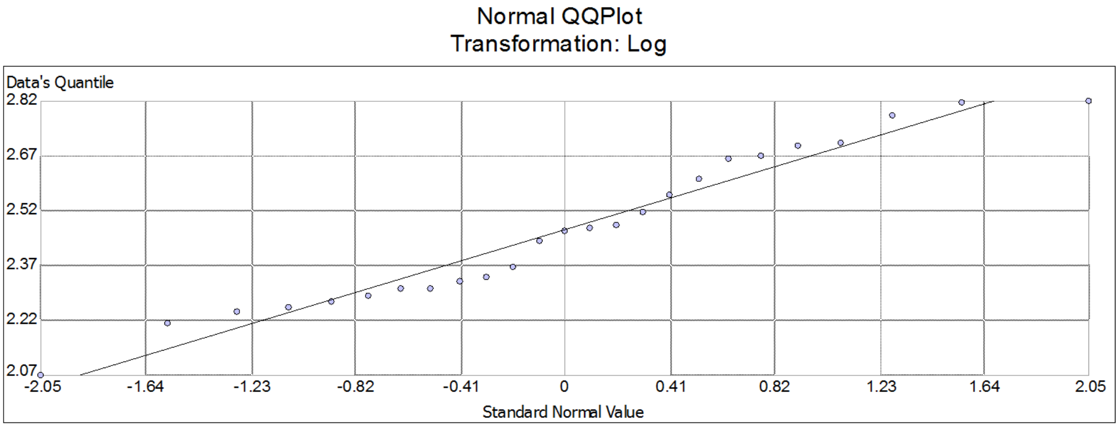

Figure 8: QQPlot(log) of GWL data (dry) in 1996

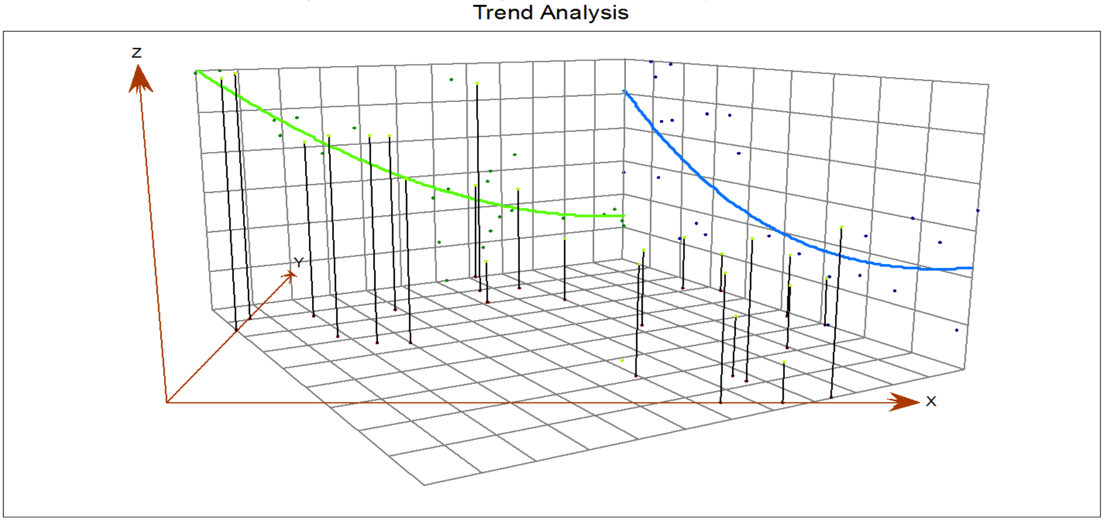

Figure 9: Trend analysis of GWL data (dry) in 1996

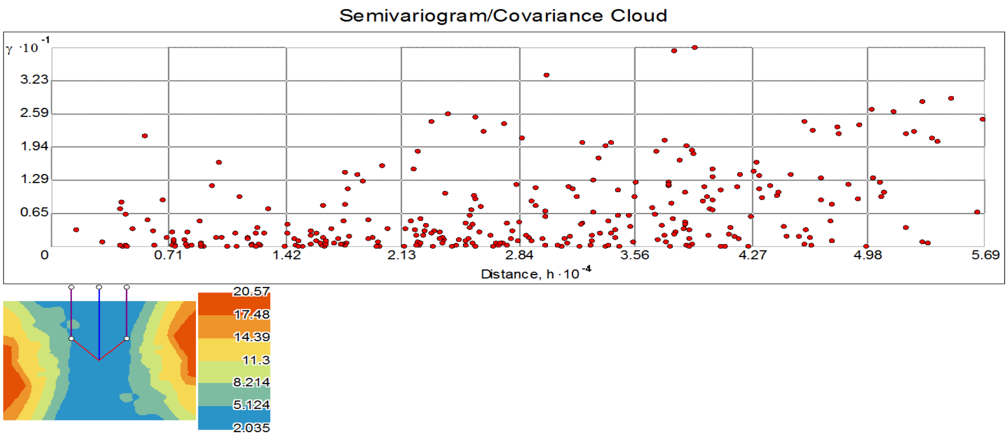

Figure 10: Semivariogram cloud for GWL data (dry) in 1996

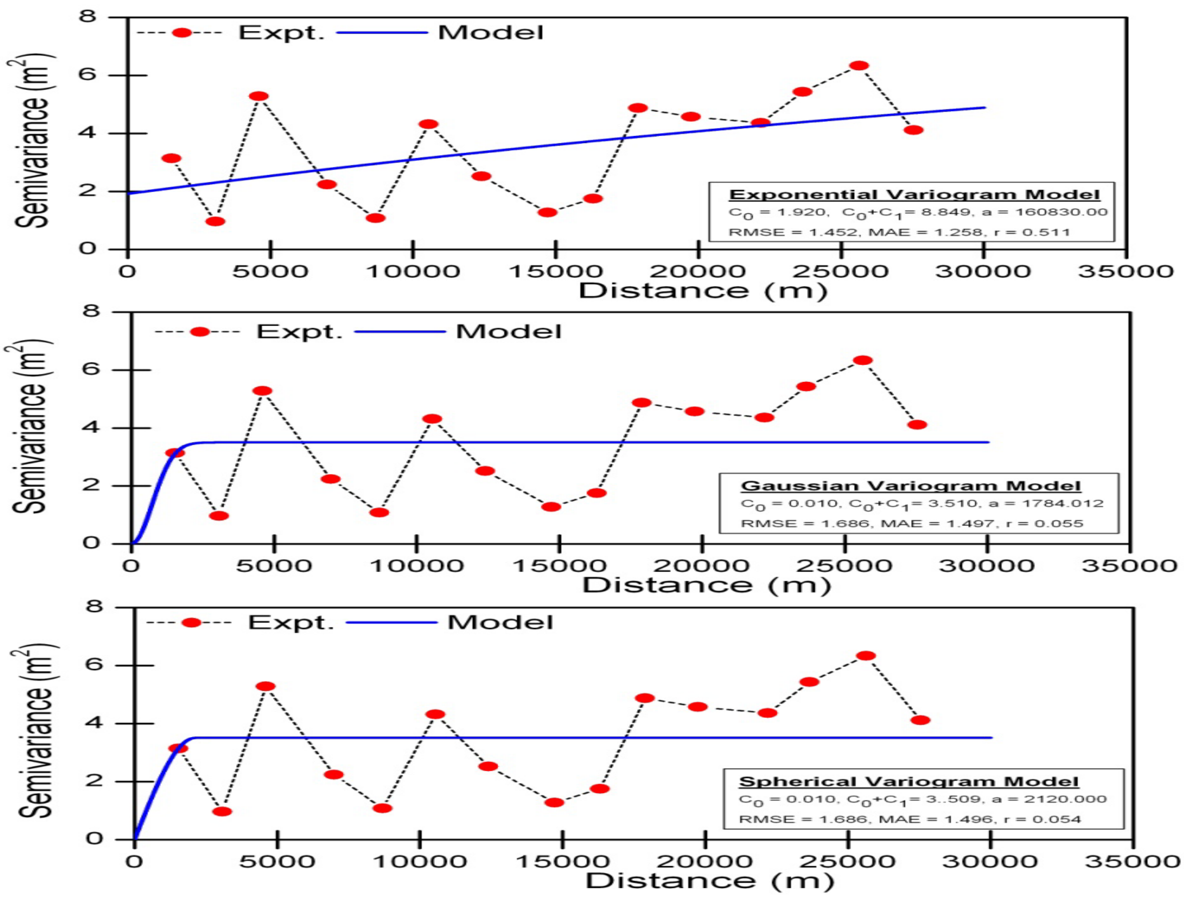

Figure 11: Different types of variograms for dry period in 1996

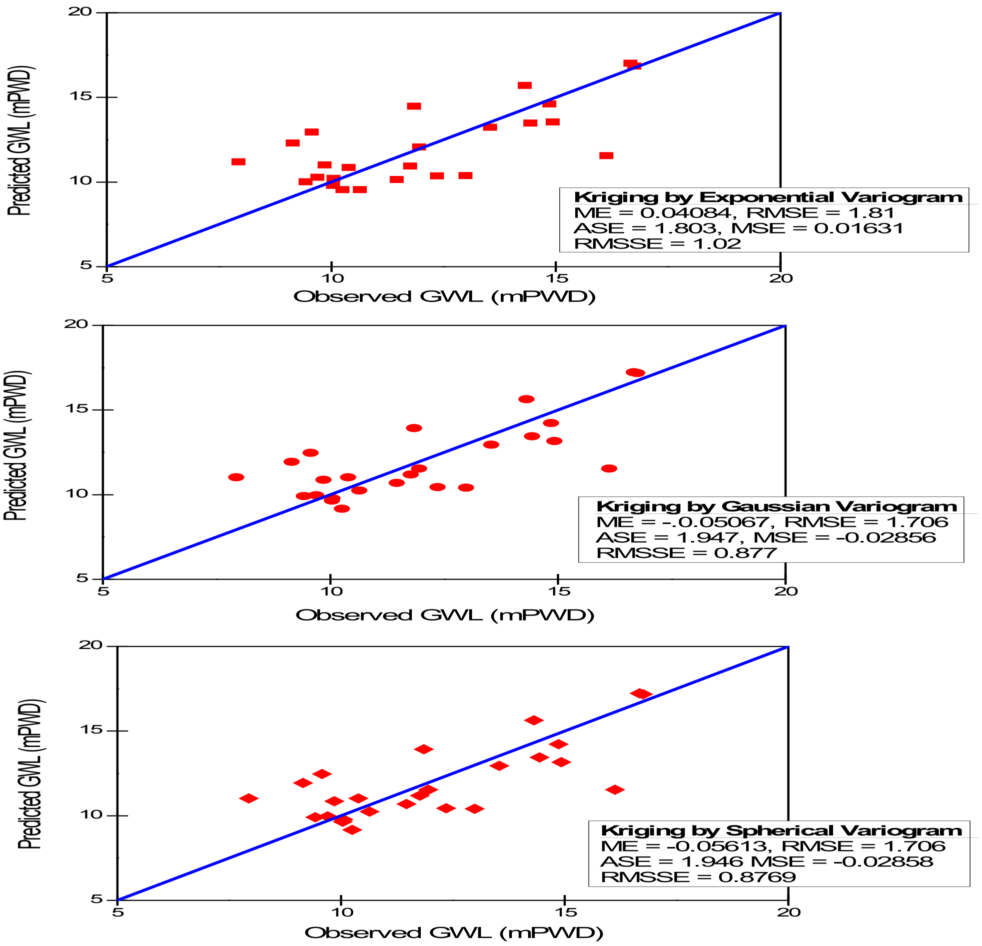

Figure 12: Cross validation for different models for dry period 1996

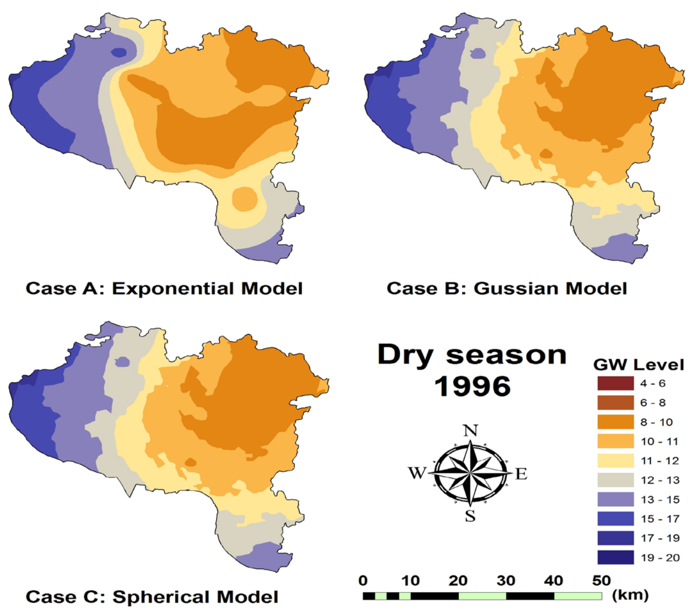

Figure 13: Surface maps of GW level for dry period 1996

3.2 Equations

The following equations are used to determine the prediction errors. Statistical prediction such as kriging provides the prediction standard error (si) for the locations.

$$\sqrt{\frac{\sum_{i=1}^{n} \hat{\sigma}^2(s_i)}{n}},$$

The mean standardized prediction error

$$\frac{\sum_{i=1}^{n} \left(\hat{Z}(s_i) - z(s_i)\right) / \hat{\sigma}(s_i)}{n},$$

And the root-mean-squared standardized prediction error

$$\sqrt{\frac{\sum_{i=1}^{n} \left[\left(\hat{Z}(s_i) - z(s_i)\right) / \hat{\sigma}(s_i)\right]^2}{n}}.$$

The root-mean-square prediction errors

$$\sqrt{\frac{\sum_{i=1}^{n} \left(\hat{Z}(s_i) - z(s_i)\right)^2}{n}}$$

3.3 Tables

Table 1: Table shows the location, coordinates and available years of data.

Well Station	Latitude	Longitude	Available data (years)
Alipur	24.49	88.71	13
Bhabaniganj	24.63	88.76	13
Bhogobontapur	24.54	88.46	13
Birkutsha	24.62	88.8	13
Dumuria	24.43	88.46	13
Gogram Hatpara	24.45	88.42	13
Gollapara	24.57	88.55	13
Gorermath	24.52	88.38	13
Gorshohurpur	24.28	88.75	13
Holidagachi	24.34	88.69	13
Jhokrakul	24.49	88.88	13
Mohanpur	24.58	88.63	13
Moktarpur	24.33	88.78	13
Moshidpur	24.28	88.8	13
Mullikbagha	24.34	88.77	13
Mundumalahat	24.62	88.54	13
Namobhadra	24.38	88.68	13
Poshunda	24.51	88.32	13
Puthia	24.42	88.83	13
Rajapurhat	24.43	88.49	13
Shamashpur	24.68	88.53	13
Sreemontapur	24.47	88.32	13
Tahirpur	24.52	88.85	13
Tanore	24.63	88.58	13
Yousufpur	24.29	88.84	13

4. CONCLUSIONS

From the analysis it is found that, the GW level is lowering in almost all the region of the study. The middle portion is sharing more than the surrounding areas. This middle portion consists of Paba, Mohanpur, Bagmara, Boalia and Durgapur. For continuing cultivation in this zone, it is essential to ensure proper recharge of GW. Irrigation required water should be managed from alternative source. Strong and effective water management works must be ensured to prevent drought and sustain the present cultivation work in Rajshahi district, northwestern of Bangladesh. The present study can be concluded with the following decisions:

- The spatial maps show the groundwater level in the study area is lowering day by day.
- This scarcity of GW is caused due to excessive extraction and dependence on GW for irrigation and other purpose.
- Lacks of surface water for irrigation is also responsible for the huge extraction of GW.
- Demand of GW is increasing for the extension of agricultural lands and cropping intensities.
- Paba, Mohanpur, Bagmara, Boalia and Durgapur area have been found as critical areas that are facing acute GW problem.
- This study suggests that less extraction and proper recharge must be ensured immediately and to find alternative water source of GW. Effective water management steps are also being inspired by this study.

REFERENCES

Alice M. (2011). Research Report on Water Scarcity in Northern Bangladesh.

Ahmeduzzaman, Shantanu K., Abdullah A. (2012). A Study on Ground Water Fluctuation at Barind Area, Rajshahi.

Asaduzzaman, M. and Rushton, K.R. (2006). Improved yield from aquifers of limited saturated thickness using inverted wells. Journal of Hydrology, Vol 326, 311–324.

Islam, M.M. and Kanumgoe, P. (2005). Natural recharge to sustainable yield from the Barind aquifer: a tool in preparing effective management plan of groundwater resources. Water Sci Technol 52:251-258.

A STUDY ON WATER LOGGING REDUCTION IN CHITTAGONG CITY

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ABSTRACT

It is credence that professional planning greatly patronage the sustainable growth of a city along with a proper drainage system consideration. Unfortunately Chittagong, the port city of Bangladesh is facing water logging on regular basis in recent years. This study is aimed to develop a draft planning to reduce water logging through enhanced understanding on the associated hydrological and hydraulic features. Commercially important CDA colony in south Agrabad, one of the most vulnerable locations facing water logging, was selected for case study. CDA colony exist in lower gradient (2.5m) compared to the elevation found at the zero point of Chittagong (i.e. 3.6m). Thus this is greatly affected by increased storm or rain, 11% increment of rain placing more vulnerability along with regular tide. This study comprised of both field study and secondary sources for data collection. Field study inclined on three selected stations to observe the flow pattern and conditions of the drains in response to tide and rain. Secondary sources accumulate information on land use, soil data, hydrological, and roads situation of the study area. Evaluating existing plans and field experiences some possible remedial measures were discussed against water logging and this is expected to be a basis for decision makers.

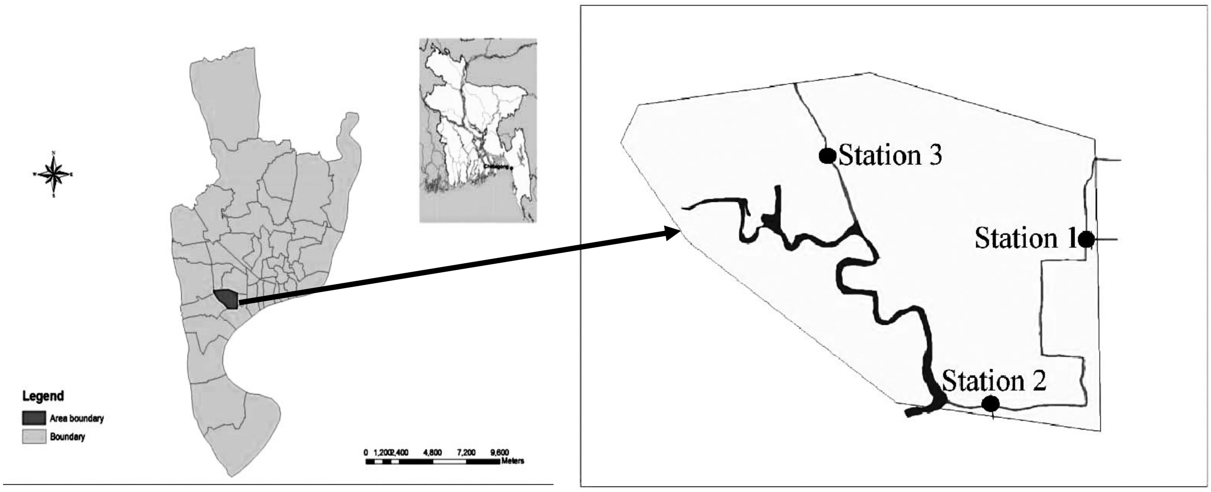
Keywords: *Water logging, existing plan for housing, drainage, hydrology, land use*

1. INTRODUCTION

For urbanization, drainage is an essential infrastructure of the modern city. According to a survey conducted by Districts Fisheries Department in 1991, the number of the water bodies in Chittagong city was 19,250 while the Physical Feature Survey conducted by Chittagong Development Authority in 2006-2007 indicated existence of 4,523 water bodies there. During the last two decades, a number of large and well-known ponds and water bodies of the city have been filled up for the construction of apartments, markets and other commercial establishments (The Daily Star, 2013). Chittagong city drains are mainly divided in three categories, are: primary, secondary and tertiary drains (CDA, 1995).The major objectives of this paper are analyzing the existing drainage condition, diversity of existing land use pattern and its impact on the water logging.
Geographic Information System (GIS) is a computer application that can be used for acquisition, storage, management, analysis of geographic information. GIS components development system could effectively manage the data such as elevation, land use and so on (Chang, 2008). We have used this application to understand the elevation, land use pattern and natural stream link of our study area through three maps.

2. MATERIALS AND METHOD

To understand the waterlogging condition we have selected three stations for data collection in CDA colony in South Agrabad (Fig. 1a). From these three stations: Bangladesh Bank Colony road (Station 1) (Fig. 1b), Abidar para (Station 2) (Fig. 1c), Islam Mia Brick field road (Station 3) (Fig. 1d), weekly data collection was carried out. Other secondary data i.e. land use, soil profile were collected from literature review, newspaper reports, websites etc. and compared with our field survey. Three types of map have created such as land use map, TIN map and stream link in this study. The land use map has been created by using the GIS data collected from Chittagong Development Authority (CDA). The TIN map is created by using the Delaunay triangulation process in Arc GIS 9.3 application. TIN is an alternative to Digital Elevation Model (DEM) and contour lines for representing the land surface. The stream link is created by several processes: (i)Firstly a filled DEM of the study area is created form the contour data using 3D analysis tool in Arc GIS 9.3. This DEM is then used to create flow direction raster by using D8 method. D8 method assigns a cell's flow direction to the one of its eight surrounding cells that has the steepest distance-weighted gradient. (ii) This flow direction raster is then used to create flow accumulation by using spatial analysis tool; and (iii) After this stream network is derived from flow accumulation raster. This derivation is based on a threshold accumulation value. Finally stream link of the study area is prepared from flow accumulation raster.



(a) Selected stations in study area

(b) Station 1 (c) Station 2 (d) Station 3

Figure 1: Selected stations

3. STUDY AREA

The location of the study area is South Agrabad, administratively designated by Ward no-27 located at the western part of Agrabad Commercial area within Chittagong City Corporation. The area stretches about 2 square kilometres. The area was covered by the Chittagong Storm water Drainage and Flood Control Master Plan 1995 under the labelled drainage area no. 4a (Fig. 2). The main watershed of this area is Moheshkhali Khal. The core zone consists of Chhota Pole Par, C.D.A. Residential Area, Abider Para, Bhohutala Colony and Bangladesh Bank Colony. The boundary is at the west and south west Mahesh Khali Khal, at the south Bandar, at the east CGS Colony and Karnafully Shishu Park and at the north Agrabad Access Road. At present this area is subject to flooding from rainstorms, this being due to siltation from the hills upstream, obstructions and poor alignment (Master plan, 1995)

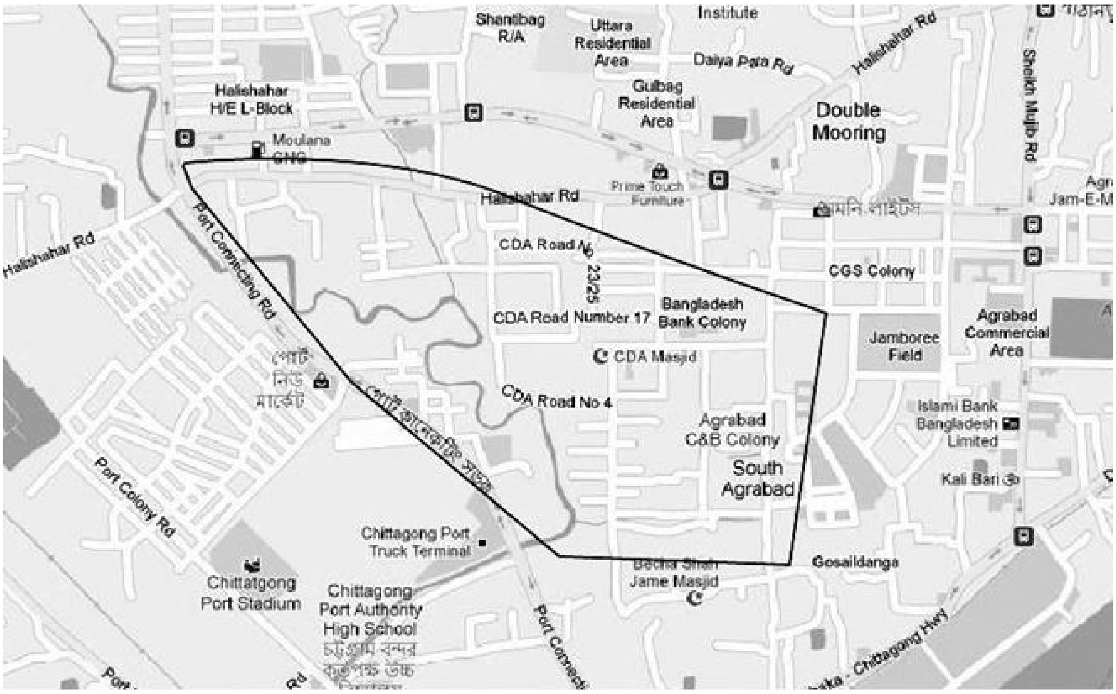


Figure 2: Study area

4. ELEVATION AND LAND USE

A Triangulated Irregular Network (TIN) map was used to understand the relative elevation of different points in our study area. The inputs to a TIN include point, line and area features. Point features are elevation points with x, y and z values. The x, y represents the location of a point and z value represents the elevation of a point (Chang, 2008). The TIN method joins the height observations together with straight lines to create a mosaic of irregular triangles. In the TIN model of a surface the vertices of the triangles produced represent terrain features such as peaks, depression and passes, and the edges represent ridges and valleys. The surface of individual triangles provide area, gradient (slope), orientation (aspect). These values can be stored as TIN attributes or can be quickly calculated when the TIN is used for further analysis (Heywood, et. al., 2005). Fig**.** 3 shows the TIN map for our study area.

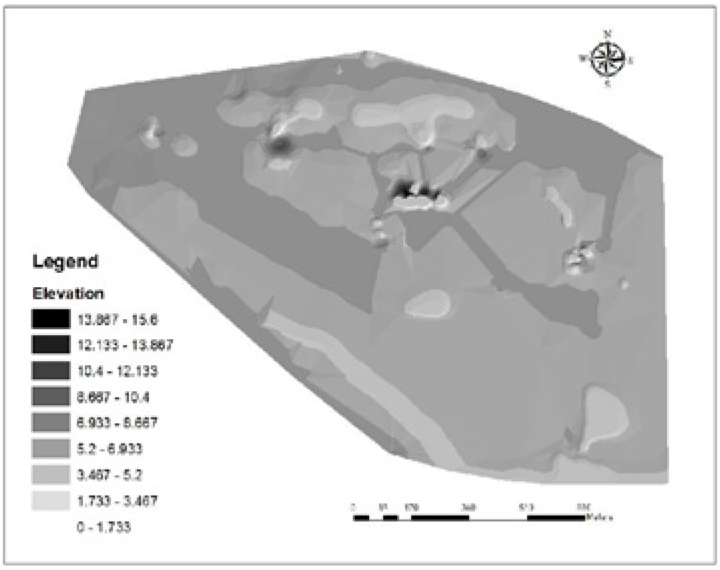


Figure 3: TIN map of study area

Allocation of right proportion of land for each aspect of the community needs is one of the most important tasks of land use planners. Improper allocation of land can lead to misuse of land. It is not only the inappropriate proportion of land for various uses but also the incompatible interrelationship of services which results in the wastage of land. As a general guide for a residential area there should be minimum 6 to 7% open space or barren land (Hamid, 1997). This will help not only in the optimum use of the area but also in the reduction of water logging. Barren lands allow the rain water to infiltrate thus reduce the excess overland flow. Figure 4 shows the land use map for our study area. We have total study area of 1312140 m^2 where barren land is 857933 m^2 (65%). So, the land use pattern of our study area is rather less responsible for water logging.

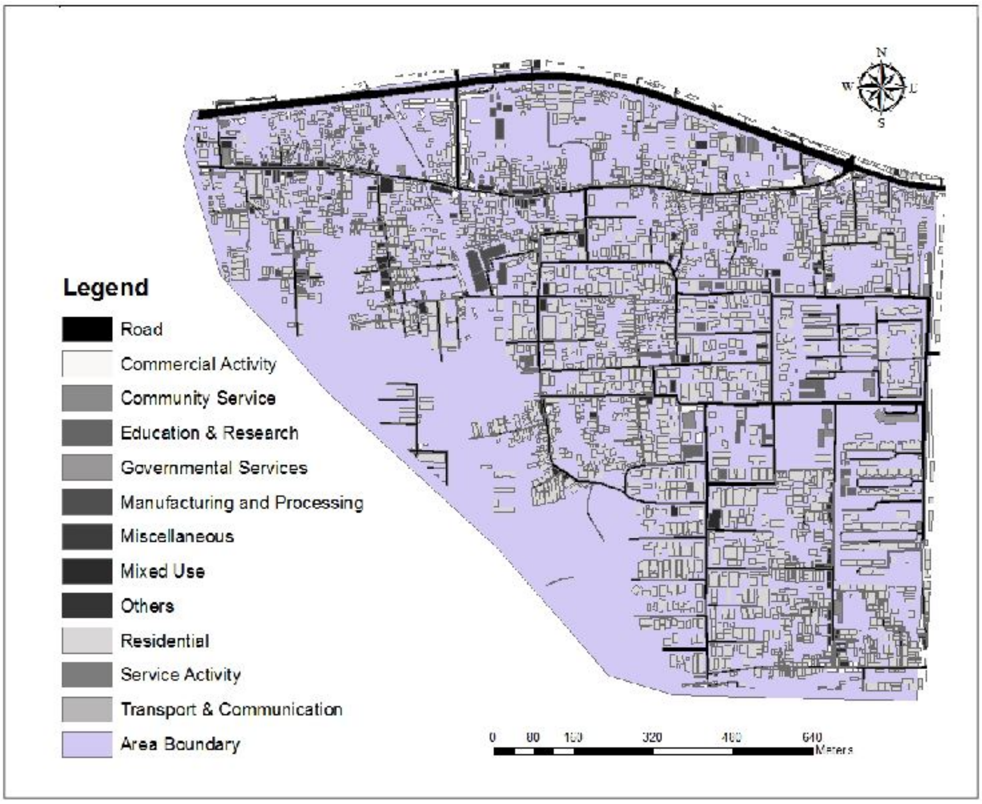


Figure 4: Land use map of the study area

5. SOIL DATA

Infiltration plays a very significant role in the runoff process by affecting the timing, distribution and magnitude of the surface runoff. The infiltration process is affected by a large number of factors but the most important one is the soil infiltration capacity. A loose, permeable, sandy soil will have a larger infiltration capacity than a tight clayey soil. Again if the top most layer of soil is a fine one then it will cause clogging during a rainfall. But vegetation reduces this process (Subramanya, 2004). The lithological data of our study area shows that the upper layer (0-4m), mid layer (4-12m) and deep layer (12-18m) consist of silty sand, dense sand and very dense silty sand successively.

6. FINDINGS

According to the field survey tidal effect is very frequent throughout out the year in the study area. The TIN map of the study area shows that there are many low lying lands which are vulnerable to this tidal effect. Field survey shows**:**

- At the time of flood tide these low lying areas experienced temporary water logging (2 -3 hours). During ebb tide though the water level goes down, still there are some places where water get logged permanently as those areas are relatively lower than the surroundings.
- The drainage system of these areas is very poor to drift the logged water.
- Because of unplanned land use the natural stream link is greatly hampered (Fig. 5).

From the soil lithological data of the study area it is certain that its condition is not responsible for water logging of this area.

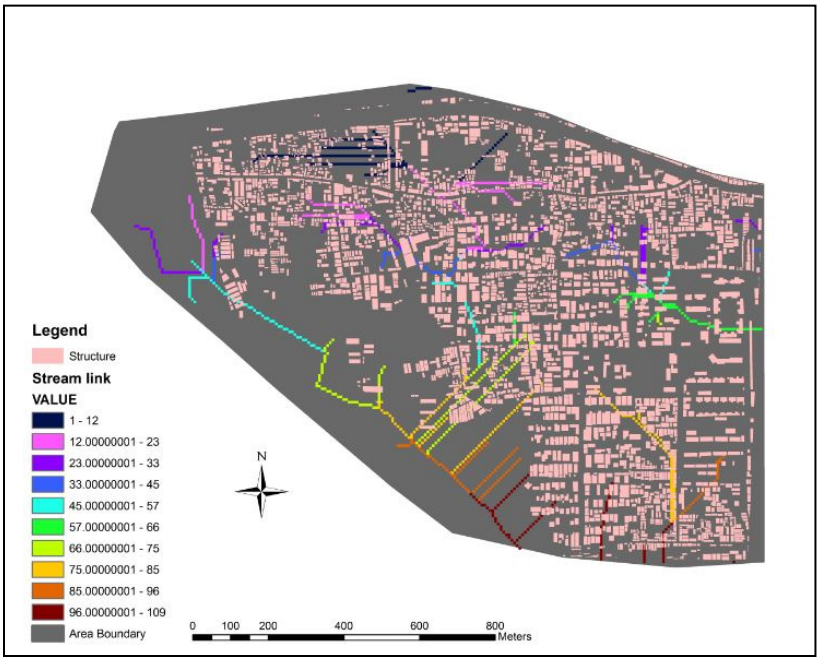


Figure 5: Natural stream network of the study area

- Form the field survey it is found that the tertiary drains' condition of this area are very poor. Many of them have no alignment, very low performance. Many drains are filled by construction materials and garbage. All these things are attenuating the overall natural stream of the study area.

Figure 6: Condition of tertiary drains

7. PROPOSED MASTER PLAN AND EXISTING CONDITION

7.1 Obstructions in the study area

It is notified in the storm water drainage and flood control master plan (1995) that the obstructions (Fig 7) i.e. corner of a boundary wall (Fig. 7a), corner of a brick wall (Fig. 7b), abutment and wing walls of a broken bridge

(Fig. 7c) should be removed from the catchment area of Moheshkhali Khal. But the present scenario is that the obstructions are still there.

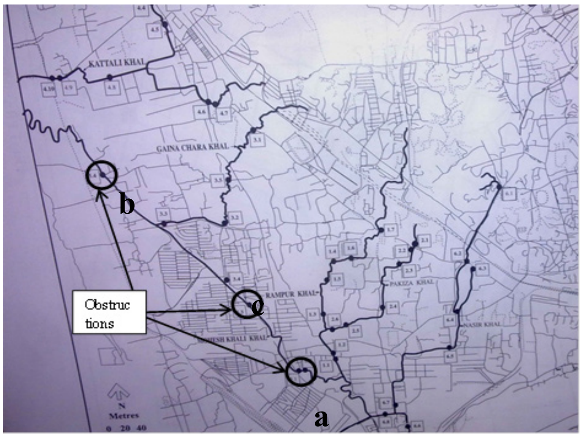


Figure 7: Obstructions of study area

(7a) Corner of boundary wall (7b) Corner of a brick wall (7c) Abutment and wing walls

7.2 Construction of the silt traps in the study area

In the storm water drainage and flood control master plan (1995) there was a provision of constructing 2 silt traps in this area. Each silt trap is divided into two compartments with sluice gates so that while water is following through one compartment the other may be emptied of silt. Emptying should start as soon as the silt level in the compartment approaches the crest level of the downstream weir. Thus there is a need to evaluate the silt trap requirement with more field survey.

7.3 Re-profiling the Primary khal

There was a provision of re-sectioning the Mohesh khal for required bed and side slopes. There was also a provision of stabilizing its side slops over its whole length which would minimize the siltation of the khal (Master plan, 1995). But it is a matter of regret that this development works cannot be implemented till now. Fig-8 shows where these works is needed.

Figure 8: **Locations** where re-profiling needed

8. CONCLUSION

Drainage planning bears great importance for a city. Collaterally this plan is also very important part in city planning. A good drainage planning can efficiently drift the unwanted water from the city. This planning encompasses transport, land use planning greatly**.** From the GIS stream network analysis, we found that the existing land use is less responsible for water logging. However, the field survey concludes existance of obstacles are causing the water logging. There is an urgent need to develop a numerical model for evaluating this scenario as well as an intensive review of the master plan to aid the decision support system.

REFERENCE

Chang, K. (2008), Introduction to Geographic Information Systems (4th edition). Tata McGraw-Hill Publishing Company Ltd. pp. 51.

Chittagong Storm Water Drainage and Flood Control Master Plan: Volume-2: Main Report.

Hamid, A. (1997). Sub-division Planning. Hamid Foundation, Dhaka-1207, Bangladesh. pp. 52.

Heywood, I., Cornelius, S. and Carver, S. (2005). An Introduction to Geographical Information Systems. Pearson Education (Singapore) Pte. Ltd., Indian Branch, 482 F.I.E. Patpargonj, Delhi 110092, India. pp. 77.

Islam, S 2013. Ponds, water bodies in Ctg city disappearing fast**.**The Daily Star dated 9th June 2013.

Subramanya, K., Engineering Hydrology (Eighteenth Reprint 2004). Tata McGraw-Hill Publishing Company Ltd. pp. 84-85.

ENGINEERING ACTIVITIES AND THEIR MISMANAGEMENT AT HALDA - A UNIQUE RIVER FOR NATURAL SPAWNING OF MAJOR INDIAN CARPS

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ABSTRACT

Halda is the only tidal freshwater river in the world from where fertilized eggs of major Indian carps are collected by the local fishermen. Besides, this river is the source of drinking water for Chittagong city dwellers, means of water transportation, irrigation, fishing activities etc. No critical scientific analysis has been conducted to date on existing engineering activities at this river. Consequently, this study aimed at critical evaluation of the management of existing engineering activities at Halda through field visits and semi-structured questionnaire based interview of stakeholders including government officials, local communities and academic authorities on Halda river. The river has been under different engineering projects and activities. The engineering activities that we have identified includes construction of rubber dams for winter irrigation, sluice gates for flood and irrigation control, sporadic and large scale (20 legal and many illegal sand extraction stations) sand quarrying by local leaders and businessmen, cutting of 11 oxbow bends over the last 100 years to straighten the river, erection of two major government irrigation projects, establishment of about two dozen of brickfields along the two banks of this river etc. All of these bear the signs of mismanagement leading to both reversible and irreversible negative impacts on the sustainable flow ecosystem services from this river. As we saw, twelve sluice gates/regulators have been erected for irrigation and drainage management of canals connecting lower Halda, among which, as the study revealed, every sluice gate has different scale of mismanagement. We suggest that the policy makers, planners and development workers should take into consideration of findings of this study to undertake any engineering related project or programmes so that both environmental sustainability and social acceptability be ensured.

Keywords: Halda river, spawning ground, engineering activities, bank erosion, sustainability

1. INTRODUCTION

Rivers play very crucial roles in the development of any country by providing a range of ecosystem services including fresh water supply, fish production, transportation, waste assimilation, recreation and tourism etc (Hitzhusen *et al.* 2000). Halda, the third main river in Chittagong region of Bangladesh after Karnaphuli and Sangu, originates from the *Haldachora* fountain located at remote Patachara hill ranges of Ramgarh upazila, Khagrachari hill district. In Bangladesh, this resourceful tidal river is the sole source of fertilized eggs of major Indian carps namely *Catla catla*, *Labeo rohita*, *Cirrhimus mrigala* and *Labeo calbasu* (Patra and Azadi, 1985; Tsai *et al*., 1981). The river provides favorable physicochemical factors for the major Indian carps and creates a congenial environment for their spawning during monsoon between April and June (Kibria *et al.* 2009; Azadi 2004a; Azadi 1985; Patra and Azadi 1985). As a tidal river, this is the only of its kind in the world from where fishermen can collect fertilized eggs directly (Kibria *et al.,* 2009). The fries from these eggs are used in fish aquaculture throughout Bangladesh to produce fresh water carps. The river also provides navigation, supplies drinking water and generates a sizeable employment opportunities for the local communities. In this way, it plays pivotal role in the local and the national income generation which makes this river a natural resource of immense economic value (Kabir, 2012). The total value of tangible resources according to our calculation stood at 20.5 million US$. Segmented contributions from fishing, fish fry, irrigation, drinking water, water transportation and sand extraction were 0.07, 0.005, 15.78, 1.33, 0.12, 2.51 million US$, respectively (Kabir, et al. 2013). Interestingly, collection of fertilized eggs, production and rearing of fish fries are conducted completely by the indigenous knowledge of the local communities which has evolved through generations through a mix of historical knowledge and religious spirit. Therefore, this river has a great cultural value as well. Day by day, however, Halda is losing its resourcefulness due to over-fishing, straightening of existing ox-bow bands, sedimentation on the river bed, sand quarrying, changes in water quality due to salinity intrusion and industrial pollution, unplanned sluice gate establishment and their mismanagement, improper brood fish management, denudation of various species of fishes towards gradual extinction – including carps, unchecked riverbank erosion and above all ecological changes caused by global climate change (Rahman and Bishwas, 2009). These problems are exacerbated by implementation of improperly planned engineering activities under different river management projects (Kibria *et al.* 2009) which have negative impacts on the sustainability of its natural resources. No scientific evaluation of engineering activities has thus far been conducted to critically analyze existing engineering interventions on this river. In order to meet this gap, this study aimed at critical analysis of the existing engineering activities. We have made a number of field visits to document the engineering activities, their management and impacts and we also have interviewed, by using semi-structured questionnaire, the major stakeholders related to engineering activities including the government officials, local communities and academic authorities on Halda river.

2. MATERIALS AND METHODS

2.1 Study area

The study has been conducted on Halda (Figure 1) which is one of the major rivers in the South-East region of Bangladesh and famous for being the breeding ground of pure Indian major carps (www.haldariver.org). Before falling into the Karnaphuli River, Halda flows through Fatikchhari, Hathazari and Raozan Upazilas and Chandgaon Thana of Chittagong city. The 98 km long river is navigable 29 km upstream (up to Nazirhat) by big boats and 16-24 km further upstream (up to Narayanhat) by small boats.

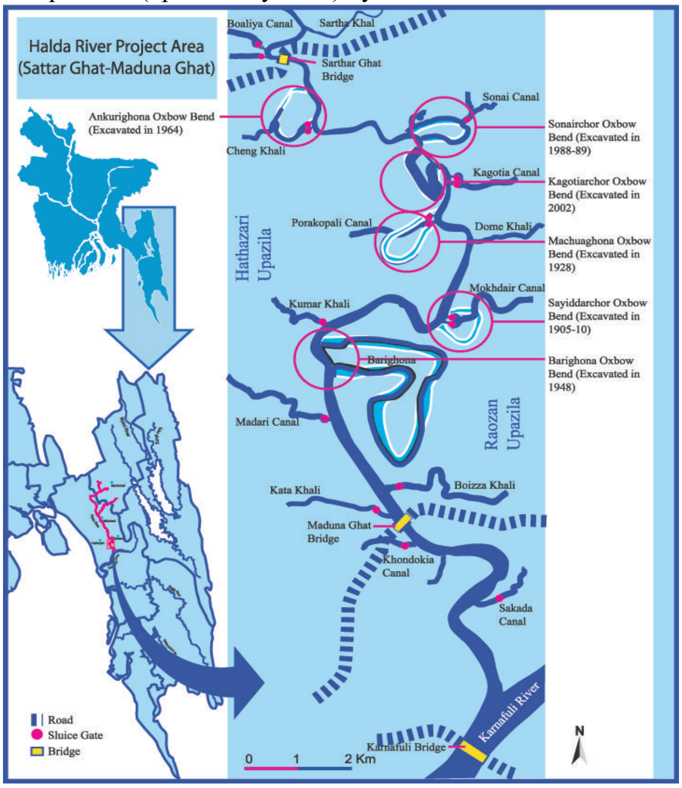

Figure 1: Map of Halda river, Chittagong, Bangladesh (Source: www.haldariver.org)

2.2 Data collection and analysis

Both primary and secondary data were collected from relevant government and non-government offices, local and national experts on this river. We have made several field visits and have conducted an extensive questionnaire based field survey in the Halda dependent community in order to collect data on existing management and impacts of sluice gates, rubber dams, irrigation projects, sand quarries, brick fields etc. Data were collected from Water Development Board (Rangamati circle, Divisional office, Chittagong, Agricultural

Extension Office, Madunaghat, Chittagong, Bangladesh) about Karnaphuli Irrigation Project (Halda Unit), Rubber dam, and Halda Extension sub Irrigation Project. Besides, a questionnaire survey was conducted among the 120 respondents from 12 villages, which was selected in such a way that two villages of along the two banks can be accessed by crossing the river with the help of boat, to assess the satisfaction towards the satisfaction of existing management of irrigation projects. Secondary data have been used to determine status and impacts of straightening of oxbow bends. Data to evaluate river bank protection due to engineering activities have been collected from Water Development Board, Chittagong which has been substantiated through field visits. In addition to this, among the 22 brickfields along the two banks of Halda up to Nazirhat, 5 brickfields including 2 large, 2 medium and 1 small, were physically surveyed. Finally, the collected data were compiled and analyzed.

3. RESULTS

3.1 Sluice gates

Sluice gates are intended for tidal and rainwater flood control as well as for water conservation to irrigate agricultural lands. A total of 12 sluice gates have been installed on canals along the lower Halda and as per our results, all of them are in dysfunctional due to mismanagement (Table 1). Consequently, one of the main reasons for siltation at lower Halda is inactive and mismanagement of sluice gates at upper stream of Halda (Figure 2). During rainy season, due to heavy rainfall, huge amount of silts accumulated from hills of upstream at the basin of the river. Besides, solid wastes block the most the sluice gates, lack of caretaker of these sluice gate, water cannot flow at its natural flow. In addition to this, through the tributaries, water cannot enter into the Halda river directly. As a result, overflow of water falls to this river by passing a number of agricultural lands. The ultimate result is river bank erosion followed by destruction of agricultural crops.

Table 1: Location of sluice gates and their existing mismanagement at Halda

Location of sluice gate	**Number of vent**	**Existing mismanagement**
Khanakia	5	No pulley, one gate was broken, water hyacinth blockade
Katakhali	4	No pulley, water hyacinth blockade, insufficient number of vents, gates were repaired
Mandari	3	No pulley, theft of some parts
Kumarkhali	2	No pulley, gates are broken, insufficient number of vents, theft of parts
Boalia	5, 7	Vent 7 is blind and 5 is blocked, Gate, pulley system and pump house is in order
Chengkhali	5	Two out of five gates are in order, operated twice a year
Poralia	8	No pulley, gates are not operable, no slab over the sluice gate
Sonaimukh	3	No pulley, non- operable gates, insufficient number of vents, water hyacinth blockade
Kagotia	4	No pulley, three out of four gates were broken , no gateman
Mogdai	7	There was no slab over the vent, the location of the sluice was so remote area
Boijyakhali	4	One out of four gate is inactive, water hyacinth blockade, operated twice a year
Sakarda	3	Two out of the gates has pulley, operated twice a year

(a) (b)

Figure 2: Mismanagement of sluice gates on canals connected to Halda (a) The control mechanism (pulley system) for gates are damaged and (b) The gates and control mechanism (pulley system) are totally destroyed, the existing structure hinders normal flow of water as blocked by debris

3.2 Irrigation activities

There are two big irrigation projects based on Halda river which are managed by the Water Development Board (WDB) and the Agricultural Extension Office of the Government of Bangladesh (GoB). The Karnaphuli irrigation project covers 18,624 hectares of land in two sub-units – Halda and Ichamati with respective coverage of 15,386 and 3,238 hectares. Ichamoti unit is beyond the scope of this research as it does not have any bearing on Halda river. In this project 14,380 hectares is irrigable of which 12,550 hectares is included in Halda unit (Figure 3). The pre-project total annual production of paddy and other crops was 35,500 MT with a projected market value of US$ 828,892.80. In the post project condition, total annual production has increased to 152,792 MT with a projected worth of US$ 21,711,072. Although the annual production of agricultural crops are irregularly increasing, but, the existing maintenance and management of this project is not upto the mark to satisfy the local peoples' expectation. Among the 120 respondents surveyed, 45% opined that they have low satisfaction about the existing management of irrigation project (Figure 4). On the other hand, Halda Extension sub-Irrigation Project of WDB covers an irrigable area of 1,820 ha at Sattarghat. Under this project, 13 water management organizations have already been formed with 12 members in each in order to ensure the incorporation of local communities into the project. The project is at pilot stage and complete implementation will take some time.

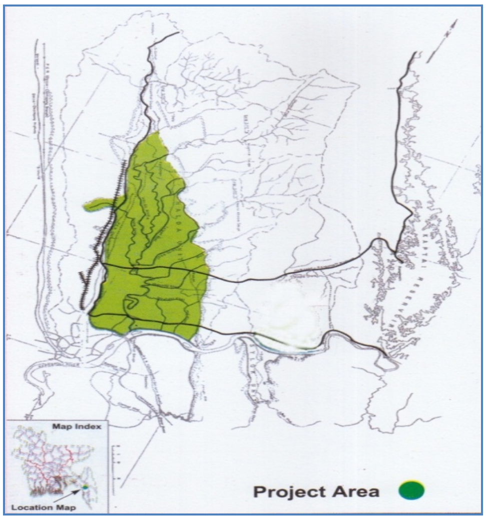

Figure 3: Irrigated area by Halda unit of Karnaphuli Irrigation Project (*Source: Bangladesh Water Development Board, Rangamati Circle*)

Besides, Local Government Engineering Department (LGED) has constructed a rubber dam at Koiyachara, Fatakchari in association with Agriculture Extension Department (Figure 5) that covers 1000 ha and benefits 1200-1500 farmers. During dry season, a large part of upper Halda becomes dry due to very low flow of water and rainfall which becomes more prominent due to the blockage of water by this rubber dam leading to accelerated siltation of the river bed. Consequently, water holding capacity of river is waning and the river fails to catch rainwater received through different tributaries.

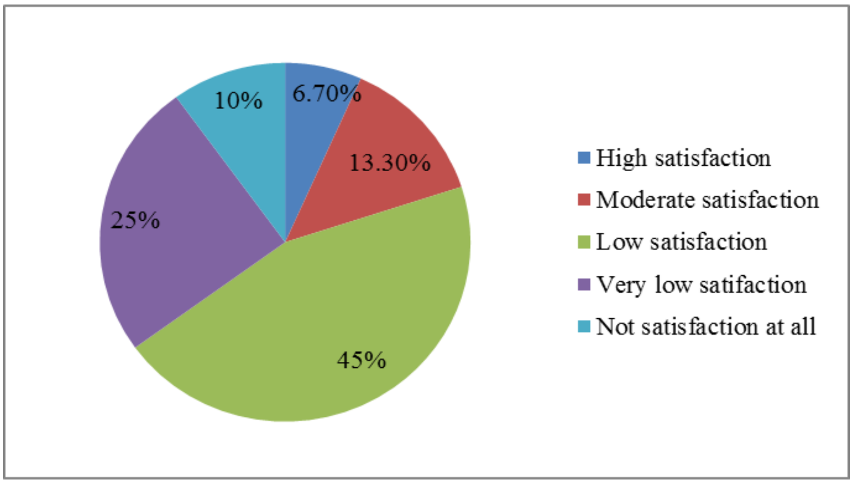

Figure 4: Local peoples' opinion about satisfaction of existing management of irrigation projects at Halda

a) Dry season

b) Rainy season

Figure 5: Rubber dam at Koiyachara, Fatakchari, Chittagong (a) The downstream side of the dam is dry and the effect of siltation is prominant (b) and the hindered flow of water during the rainy season

3.3 Sand extraction

Legal and illegal sand query from Halda is a very profitable business. Legal *Balu Mohals* (sand stations) are leased to individuals through competitive bidding by the Revenue branch of Deputy Commissioner's office of Chittagong district. We have identified all legal and illegal sand extraction stations at Halda (Figure 6 and 11). There was higher number of illegal sand selling stations than legal *Balu Mohals* (Table 2). Besides, at lower Halda at least five heavy dredgers with average carrying capacity of 4000 cft sand have been extracting sand three times every day. Total annual economic benefit from sand extraction was 2,506,157 US$ (Kabir et al. 2013). Sand quarry, which is the main factor behind river bank erosion, is also damaging the river basin and affecting the whole river ecosystem.

Table 2: Legal and illegal Balu Mohals (sand querying stations) at Halda river

List of Balumahal of Halda river	Illegal Balu Mohal of Halda River
1. Koiyaghat BaluMohal, Fatickchari	1. ModunaghatbaluMoohal
2. New Bridge BaluMahal, Farhadabad, Fatikchari	2. Sattarghat Bridge (South) BaluMohal
3. Halda river-4 BaluMohal, fatikchari	3. Sattarghat Bridge (West) Balu Mohal-1
4. Halda river-1 BaluMohal, Chandpur	4. Sattarghat Bridge (West) Balu Mohal-2
5. Brahman Halda river BaluMohal, Fatikchari	5. Baidder Hat
6. Halda River AzimChowdhuryGhat, Fatikchari	6. Naya Hat
7. SattaKhalBalu Mohal-3, Raozan	7. SugolchariBaluMohal, Fatikchari
8. Satta khal-1 Balumahal, Gohira, Raozan	8. Panchpukuria (Nasimerghat, Fatikchari)
9. HaldaBalu Mohal-2, Shoillakopa, Fatikchari	9. Bhangar pool, Fatikchari

10. Halda river GolaPukur, Fatikchari	10. Mirzar hat, Fatikchari
11. AkKhuliaBaluMohal, Fatikchari	11. Nanar hat, Fatikchari
12. RuhulapurBaluMohal, Nazirhat	12. Jhunirghat, Fatikchari
13. Nazirhat College gate baluMohal	13. Eunoserghat, Fatikchari
	14. Jhuggar Chula, Fatikchari
	15. AzimchowdhuryerGhat, Fatikchari
	16. South Sadan Nagar, Fatikchari
	17. Mohora
	18. AzimerGhat
	19. Kagotia
	20. West binajhori

(a) (b)

(c) (d)

Figure 6: Sand quarrying at Halda river (a) The heavy duty dredgers in operation to collect sand from midline bottom of the riverbed (b) The vessel filled up with sand (c) The collection of sand from the bottom near the bank of the river (d) The resulting river bank erosion

3.4 Straightening of oxbow bends

Over the last 100 years since the beginning of 19th century, 11 oxbow bends, eight at lower Halda and four at upper Halda river, have been straightened (Figure 7) to reduce the total length of the river from 123 km to 98 km (Kibira et al., 2009). In 2002, the Kagotia char oxbow bend near Gorduara was straightened by digging a 10 feet wide canal and it was subsequently cut again in 2004, 2007 and 2009 which substantially hampered the natural flow of this river. As a result, heavy water flow during rainy season hits the adjacent banks. Since the length of river is reduced by about 25 km, the water holding capacity is also reduced leading to water overflow with a great velocity during rain causing severe erosion to river banks of Halda. Also, the river's ability to support brood mother fishes of major Indian carps is related to the ox-bow bends. The straightening of these bends is severely affecting the ability of the river to produce eggs and fries of major Indian carps.

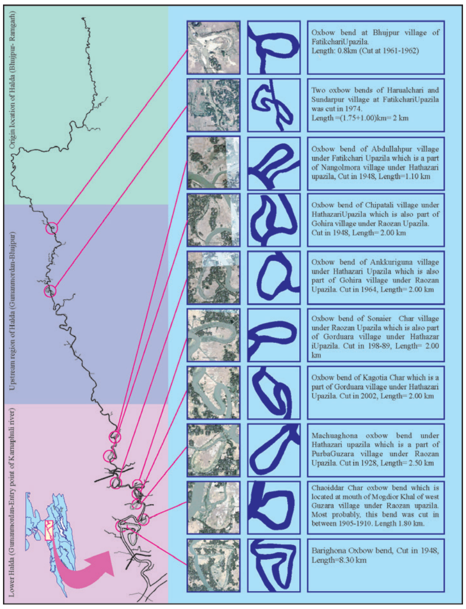

Figure 7: Cutting of oxbow bends at Halda river over the last 100 years (Source: Kabir, et al. 2013)

3.5 Protection of river bank

Bangladesh Water Development Board (BWDB), Chittagong has protected around 1.5 km river bank by setting concreted blocks at some erosion prone areas (Table 3 and Figure 8). But it is very much negligible corresponding to the requirement. Considering existing mismanagement and inadequacy, local communities have applied their indigenous knowledge using the locally available resources at a very small scale to protect the banks of this river (Figure 8).

(a) Government initiative and its mismanagement

(b) Local community’s initiative

Figure 8: Protection measures of river bank erosion and their mismanagement at Halda, Chittagong, Bangladesh (a) the concrete blocks placed by the WDB and (b) Local efforts to prevent river bank erosion

Table 3: Protection measures of Halad river bank taken by government of Bangladesh (Source: Bangladesh Water Development Board, Chittagong

Location	Length (meter)	Present status
Uttar Mekhol	1150	Good, 150m ongoing
Peshkarhat	350	Not good
Aburkhil	130	Good
Dakshin Madarsha	400	Ongoing

3.6 Brickfields

We observed 22 brickfields along the two banks of Halda river up to Nazirhat (Kibira et al., 2009). Our survey revealed that these brickfields collect soil to make bricks from this river and the river also acts as a mean of cheap water transportation for supplying the bricks. Also, these brickfields use the water from this river during brick processing (Figure 9 and 11). All these activities are adding to the erosion of the banks of this river

Figure 9: Brickfields at bank of Halda river, Chittagong, Bangladesh

3.7 Others engineering activities

The study found that some other engineering interventions i.e. bridges, culverts, hatcheries etc adjacent to Halda river. We identified 14 bridges at different locations across this river (Figure 10 and 11). Unexpectedly, we also found that some broken culverts and roads, poorly managed hatchery along the two banks of Halda (Figure 10).

a) Bridge

b) Broken culvert

c) Hatchery

d) Broken road

Figure 10: Impact of different engineering structures on the erosion of Halda river's bank

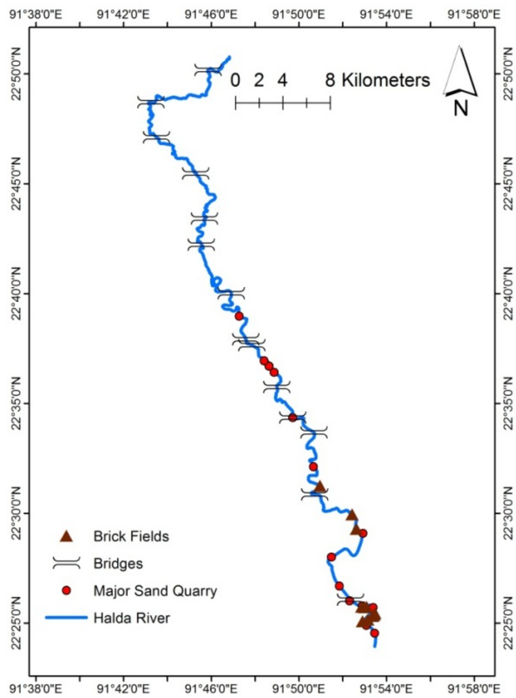

Figure 11: Geo-location of brickfields, bridges and major sand query stations at Halda river

4. CONCLUSION

Halda river is one of the most resourceful rivers in Bangladesh. This river provides a number of products and services all-round the year to the communities living in its vicinity. However, it has attained a very special identity and is frequently referred to as a national natural heritage for being the only breeding and spawning ground for major Indian carps. This identity has brought this river under the spotlight of both national and international interests. Through this research, our effort was to identify the management status of existing engineering activities. In the study, we found that the flood and water control structures are unmanaged or poorly managed, for example, each of 12 sluice gates/regulators has different sorts of dysfunction. Irrigation projects, unplanned and excessive legal and illegal sand quarrying, straightening of ecologically significant oxbow bends, erection and operation of about two dozens of brickfields along the two banks of this river are the major engineering structures and operations. None of these projects and structures are planned by keeping in mind the ecosystem of the river and the ecosystem services that local people enjoys from it. The mismanagements of these engineering activities are negatively affecting the sustainability of its natural resources. We suggest that the policy makers, planners and development workers should take into consideration of findings of this study to undertake future engineering project and reevaluate the existing interventions.

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REFERENCES

Azadi, M.A. (2004a). Management of spawn fishery of major carps (*Labelorohita*, *Catlacatla* and *Cirrhinuscirhosus*) in the Halda River, Chittagong, Bangladesh. Bangladesh J. Fish (Special Issue), 27: 50-51.

Bangladesh Water Development Board (BWDB), (2009). Karnaphuli Irrigation Project, palmlet report, pp. 8.

Hitzhusen, F.J., Sarah, L., Ayalasomayajula, R. (2000). 'Muskingum River Economic Valuation', Department of Agricultural, Environmental and Development Economics, The Ohio State University.

Kabir, M.H., Kibria, M. and Hossain, M.M. (2013). Economic valuation of the irrigation water use from Halda - a unique river of Bangladesh, European Journal of Agricultural Sciences, Volume 10, 2013, pp. 54-63.

Kabir, M.H., Kibria, M.M., and Hossain, M.M. (2013). River bank erosion and its protection at Halda: a unique river from south-eastern part of Bangladesh, Proceedings of the International conference on climate change impact and adaptation-2013, 14-16 November 2013, DUET, Gazipur, Bangladesh, pp. 537-544.

Kabir, M.H., Kibria, M.M., Jashimuddin, M. and Hossain, M.M. (2013). Economic Valuation of Tangible Resources from Halda– The Carp Spawning Unique River Located at Southern Part of Bangladesh, International Journal of Water Research, 1(2): 30-36.

Kabir, M.H. (2012). Economic valuation of Halda river in Chittagong, MS thesis, Institute of Forestry and Environmental Sciences, University of Chittagong, Bangladesh, pp. 111.

Kibria, M., Farid, I, and Ali, M.(2009). 'Halda Restoration Project: Peoples Expectation and Reality', Chittagram Nagorik Uddogh, Chowdhury Mansion, Polytecnic Road, Nasirabad, Chittagong. pp 57.

Patra, R.W.R., and Azadi, M.A. (1985). Hydrological conditions influencing the spawning of major carps in the Halda River, Chittagong, Bangladesh.

Rahman, M.A. and Bishwas, N. (2009). Halda project: Causes of reducing carp spawning and development of spawning during the season. B. Sc. (Hons.) project paper, Department of Civil Engineering, Chittagong University of Engineering and Technology, Chittagong, Bangladesh. pp 64

Tsai, C., Islam, M.N., Karim, M.R., and Rahman, K.U.M.S (1981). Spawning of major carps in the lower Halda river, Bangladesh, Estuaries 4(2): 127-138

www. haldariver.org (Last accessed on 21 November 2013)

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HAZARD ASSESSMENT FOR FLASH FLOOD IN DEEPLY FLOODED HAOR BASIN OF BANGLADESH

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ABSTRACT

The total area of Sunamganj district, some part of Habiganj, Moulavibazar, Kishoreganj and Netrakona districts are covered by major Haor Systems in the north east region. Flash flood is the most commonly occurring water related disaster in the haor area. Flash flood usually occurs during the pre-monsoon season (March-April-May). During the flash flood it is very common that people lost their primary agricultural productions. Annual maximum and pre-monsoon maximum water level data were used for flood frequency analysis. Inundation information is extracted from an existing 2D hydrodynamic flood model (MIKE 21) for 2Year, 10Year, 20Year and 100Year recurrence intervals. Arc-GIS are used to generate the flood hazard map. Flood hazard map is developed considering flood extent, inundation depth, duration, and flow velocity. Hazard map for the combination of flood depth and duration is prepared by designing hazard magnitude. Where duration is taken as dependent variable, flood depth and extent is taken as independent variable. Critical date is fixed at 120 Julian days of year for the determination of duration. From the hazard assessment it is found that, flood area increased with flood intensity. Higher flood depth increased and lower flood depth decreased with an increase in flood intensity. Flooding of cultivable land indicated potential damages in food production and negative effects on the livelihoods.

Key Words: *Flash Flood, Frequency Analysis, HM, Hazard Map*

1. INTRODUCTION

Hazard means the occurrence, within a specific period of time in a given area, of potentially damaging natural or man-made phenomena. Hazard assessment is concerned with the characterization of the nature, magnitude and timing (including frequency and duration) of hazard events. One of the steps involved in assessing a hazard is its detection and measurement. Techniques of hazard assessment are monitoring and diagnosis. The first one is 'Monitoring', which observes, records and analyzes products, processes or phenomena for the recurrence of hazardous events. Siddiqui and Hossain (2006) investigated that, flood damage during 1988 was the maximum, though the flood frequency of 1998 flood was almost double than 1988 flood in Bangladesh. In the recent years, for example, during the 2007 flood, although the flood was represented as a medium category flood but the devastation in the agriculture sector was huge mainly due to repeated occurrence of flood in a short interval. Another step in assessing a hazard is 'diagnosis', which is the identification of hazard through analysis of indicator.

2. FREQUENCY ANALYSIS

Flood frequency analyses of the observed pre-monsoon maximum water levels and discharges at the eighteen selected stations in north-east region were done to determine the return periods. Normal, Log Normal, Exponential, Gamma, Pearsion type III, Log-Pearsion type III and Extreme Value Type I distribution functions were tested to fit the observed data. The distribution functions were selected based on the chi-squared test. The dimensionless hydrographs were developed from the observed hydrographs of different flood years. A 2-D hydrodynamic model (Mike 21) was used to simulate the flow of water in the haor basin to get the computed water levels at an interval of 15hr in order to determine the flood depth, duration and velocity in the flooded areas. Pre-monsoon maximum and annual maximum water level and discharges for different return periods, along the Surma River close to the study area are shown in Figure 1 & Figure 2.

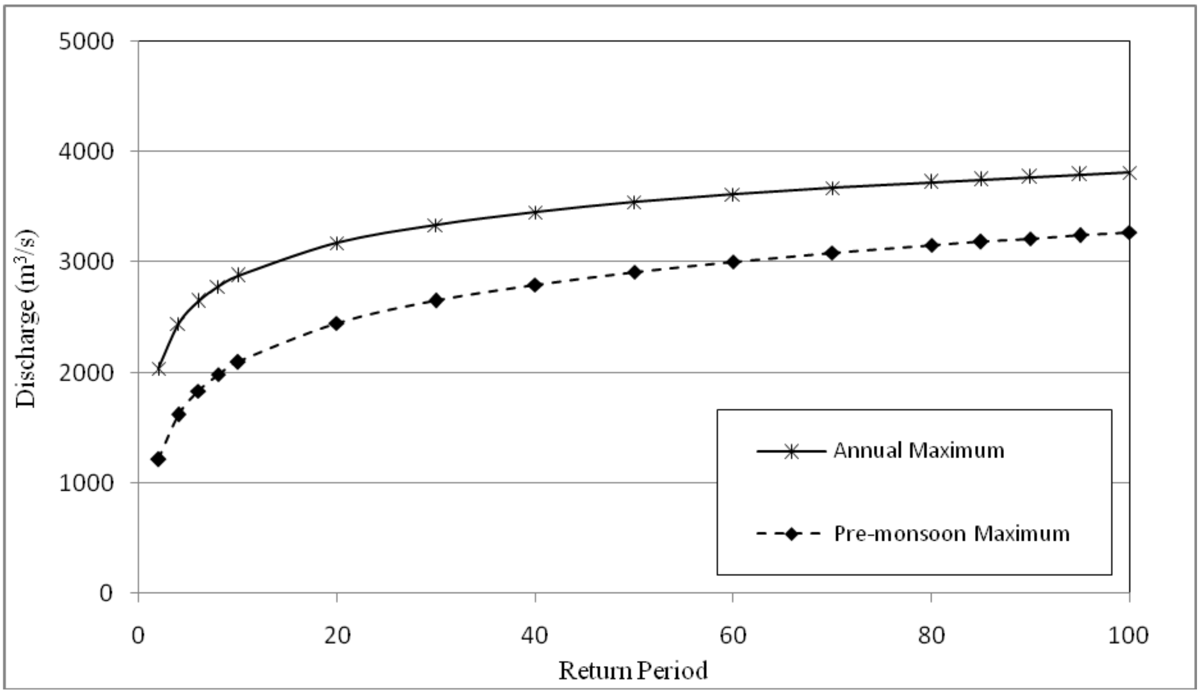

Figure 1: Pre-monsoon maximum and annual maximum discharges with the return period (Station ID 269)

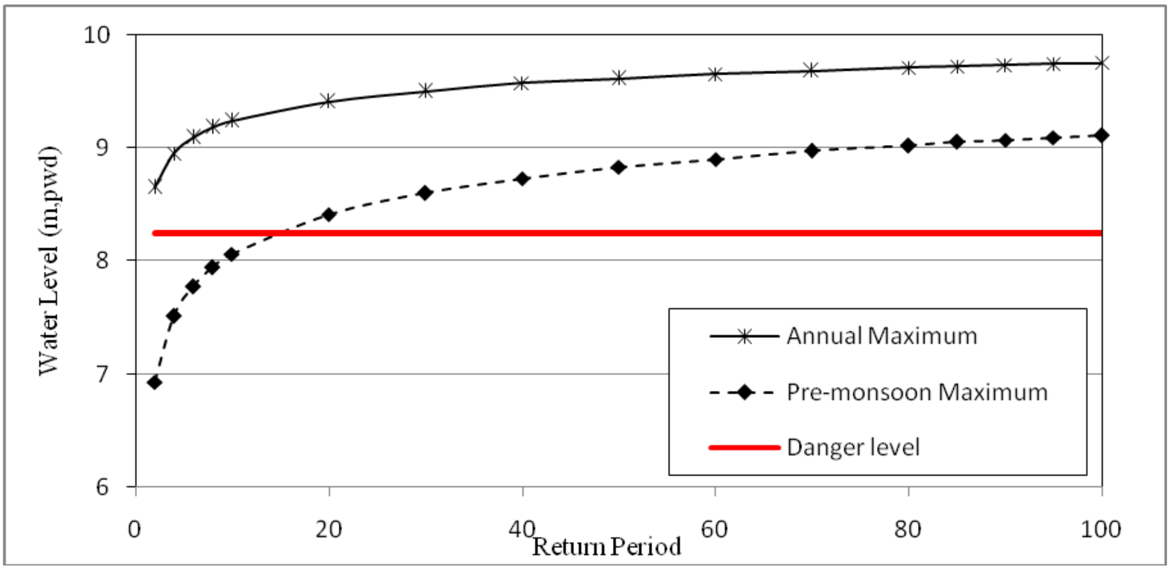

Figure 2: Pre-monsoon maximum and annual maximum water level with the return period (Station ID 269)

The pre-monsoon maximum water level at each computational point was determined from the computed water level at different return period flood events through frequency analysis. Annual maximum water level was determined for the comparison with the pre-monsoon maximum. The depth of flooding in each grid point was calculated by subtracting land elevation from the computed flood level. To generate the inundation map of flood, Arc-GIS software was used. Inundation depth information for specific duration is used in Arc-Catalog to create XY point feature class. All point feature class is used in 3D-analyst tool in order to interpolate raster through the inverse distance weighted interpolation method. Grid cell size of 150 m by 150 m is used. Flood inundation map is prepared from the spatial analyst tool by using raster map as input raster and the map as input mask feature through extracting mask tools in the Arc-GIS. The generated flood inundation map is used for preparing hazard map.

3. HAZARD MAP

In the present study hazard is identified due to flash flood in the haor basin. Different frequencies of flood events were considered. When an event creates potential exposure to human intervention exceeding a significant level of threat then it is said to be a hazard of that locality. The flood hazard maps include historic as well as potential future flood events of different probability. The maps illustrates the intensity and magnitude of hazard in a selected scale and are at the basis of considerations and determinations in land use control, flood proofing of constructions and flood awareness and preparedness (Alphen and Passchier, 2007). Hazard maps were prepared by considering combination of two hazard parameters. In this combination depth and duration were considered, where depth of flood is taken as independent variable and flood extent and duration is taken as dependent variable.

3.1 Hazard Parameters: Depth and Duration

The flood hazard is categorized based on the level of consequences to daily life and/or damage to properties. In general, flood hazard always classified as low, medium and high (UNDRO, 1991; Tingsanchali & Karim, 2005). The intensity of flood hazard can be represented by a relative scale, which represents the degree of hazard and is called a hazard index (HI). In this study, the relative scale of each category is considered which is similar to that of Tingsanchali and Karim (2005). A sensitivity analysis was performed by Karim and Chowdhury (1995) for three alternative scales, namely (i) HI increases linearly, (ii) the rate of increase in HI is linear and (iii) HI increases geometrically. They found that the linear scale is the best option. In this study, linear scale is considered to represent hazard index for depth and duration parameters of flood.
By using the inundation map, hazard maps were developed by selecting four hazard categories based on agro-ecological zone land class. Considering the land use characteristics of the study area, it is important to categorize hazard based on different land level used by different cropping pattern. Agro-ecological zone land classes were recognized on the basis of hydrology, physiography, soil types, cropping patterns, and seasons[1]. Besides, an agro-ecological zone indicates an area characterized by homogeneous agricultural and ecological characteristics. The agro-ecological zones have been identified on the basis of four elements such as physiography, soils, land levels in relation to flooding and agro-climatology.
Physiography forms the primary element in defining and delineating the agro-ecological regions. Soils form the second element in defining and differentiating agro-ecological zones as soil conditions determine important properties for plant growth, moisture supply, root aeration and nutrient supply. The third factor is land level in relation to flooding. In this regard, the haor basin has been classified according to agro-ecological zone land classes[2] into four types of land levels in order to delineate four hazard categories such as:

a. Medium highland, F1: land which is normally flooded up to 90 cm deep during the flood season
b. Medium lowland, F2: land which is normally flooded between 90 cm and 180 cm deep during the flood season
c. Lowland F3: land which is normally flooded between 180 cm and 300 cm deep during the flood season
d. Very lowland, F4: land which is normally flooded deeper than 300 cm during the flood season

An additional class, bottomland, is recognized for depression sites in any land level class which remains wet throughout the year. In the haor basin this bottomland are also used for local variety of boro rice in the dry season. This variety can be harvested within short period of time. In the fourth factor named agro-climatology, the suitability of season and land for a particular crop type is considered.
A smaller hazard index was assigned for a lower depth or low hazard, while a larger hazard index was used to indicate a higher hazard. The hazard index for agriculture is represents here according to above four land classes by an integer value for hazard index (HI) and it is observed that HI increases linearly in the case of agricultural crop damage. The hazard index values for different categories of flood depth are given in Table 1.

Table 1: Hazard Index values for different categories of depth of flooding

Depth Classes (m)	Definition	Hazard Index (^{d}HI)	
0-0.9	Low	1	$^{d}HI_1$
0.9-1.8	Moderate	2	$^{d}HI_2$
1.8-3.0	High	3	$^{d}HI_3$
>3.0	Very High	4	$^{d}HI_4$

For the duration of flooding there is no critical duration as the damage function varies with the type of land use properties. It is, however, doubtless that the damage increases with duration and consequently classification of flooding duration categories is desirable. There is no standard time limit to define the hazard category for duration of flooding. In the present study, four categories of duration 1day, 3days, 7days and >7days of flooding were used. These four categories of duration were used to collect field information. Moreover, local people were very much interested to provide previous crop damage information according to 1day, 3days, 7days and more than 7days duration of flooding. Four categories of duration of flooding 1day, 3days, 7days and larger 7days has dealt in conventional terms as short, medium, long and very long respectively. The term 'short duration' represents the insignificant damage of standing crops, while the term 'very long duration' represents the complete damage of agricultural productions. A linear scale of HI for flooding duration similar to the scale of depth of flooding was used. The hazard index values for different categories of duration of flooding are given in Table 2.

Table 2: Hazard Index value for different categories of duration of flooding

Duration Classes (day)	Definition	Hazard Index (${}^{du}HI$)	
0-1	Short	1	${}^{du}HI_1$
1-3	Medium	2	${}^{du}HI_2$
3-7	Long	3	${}^{du}HI_3$
>7	Very Long	4	${}^{du}HI_4$

3.2 Determination of Duration From Depth and Flood Extent

Depth of flood water in haor basin during pre-monsoon flash flood rises abruptly. Rate of rise of water depth is not always the same as the increase of flood stage. When aerial extent of flood water increases from lower stage to a higher stage, the water depth is falling down for different ranges of flood depths. Similarly, if aerial extent is larger than previous flood area for different depth ranges, then the effective depth of flood has to be lower. As discussed earlier, duration of flood water is determined from the four categories of flood depth ranges. It is important to mentioned here that, flood affected area in haor basin is mainly used for agricultural boro crop. From the field survey and secondary source of information, the critical date of harvesting has been fixed at the end of April, i.e., 120 days of Julian date of a year. Four areal extent of flood were determined from the four depth ranges. From these areal extents, the maximum area has been selected as a reference flood extent. Using this reference flood extent, aerial variation at different ranges of flood depth has been determined as a functional value, f_s. Average rate of rise of flood water 'r_s' is determined from the maximum depth of flooding divided by total duration of flooding for an event.

From the Mike21 model data analysis, it has been found that, for 2Year, 10Year, 20Year and 100Year return period flood event, the maximum depths of flooding up to the critical date till the end of harvesting are 2.68m, 5.2m, 7.72m and 11.43m respectively. Also total flooding durations for the corresponding depths of flood are 945hour, 600hour, 240hour and 495hour respectively. Now, the average rate of rise of flood water is determined from the following formula,

$$r_s = \frac{d_{max,i}}{T_{du,i}} \tag{1}$$

Where,

$d_{max,i} = Maximum\ depth\ upto\ critical\ date\ of\ event, i$

$T_{du,i} = Total\ duration\ of\ flooding\ upto\ critical\ date\ of\ event, i$

Duration of flooding is determined from the flood depth classes. There are four classes of flood depth which are used to classify the land area with the magnitude of flood impact. Each depth class has a flood extent for different duration of flooding. Effective depth of flooding that change with the variation of flood extent is assessed at a critical date of harvesting of crops for different ranges of flood depths. After the classification of flood extent according to different depth classes, duration of flooding is determined from following equation:

$$Depth\ ranged\ maximum = Duration \times (r_s \pm f_s) \tag{2}$$

Depth ranged maximum is determined from each range of flood depth. Duration is determined in the form of day such that the final quantification and qualification of flood duration is easily determined. The term 'f_s' is represents a functional parameter of depth, which rises with the variation of flood extent and duration. From the total flooded area, duration and maximum depth at a critical date, the value of 'f_s' is determined in m/hr. The 'f_s' is determined in such a way that, the adjustment of flood depth either falls or rises due to variation of flood extent with the flood depth classes. It is determined by using the following formula,

$$f_s = \frac{d_{max,i} \times a_s}{A_t \times T_{du,i}} = r_s \times \frac{a_s}{A_t} \qquad (3)$$

Where,

$a_s = Aerial\ change\ compared\ to\ lower\ stage$

$A_t = Total\ area\ covered\ during\ flooding\ period\ at\ a\ critical\ date$

Rate of rise of flood water is not always uniform. Because land elevation at all locations of flooded area is not always at the same level. For the variation of land level, depth of flood water is a function of the flooded area. Flood duration for different depth ranges is determined from equation 2. In equation 2, average rate of rise of flood water is adjusted with the fall or rise of flood depth in the next higher stage of flood. When aerial extent is positively increased at the upper depth ranged from the lower, then the adjusted functional parameter, f_s is added with the rate of rise, r_s. If the aerial extent is decreased compared to the lower depth ranged covered area then the adjusted functional parameter is subtracted from the rate of rise of flood water. For the development of flood hazard zone in the haor basin, five depth ranges were selected in this study which was discussed earlier.

Different land areas may have different depth and duration. Firstly, flooded areas were classified with respect to depth, and then of each land area is categorized by using flood duration. In order to set a hazard index for a particular land from the combination of depth and duration of a flood event, higher hazard category has been prioritized for each land area. Possible combinations for depth and duration of hazard are shown in Table 3.

Table 3: Possible combinations of depth and duration of a flood event

Depth (m) \ Duration(day)	0-1	1-3	3-7	>7
0-0.9	0-0.9, 0-1	0-0.9, 1-3	0-0.9, 3-7	0-0.9, >7
0.9-1.8	0.9-1.8, 0-1	0.9-1.8, 1-3	0.9-1.8, 3-7	0.9-1.8, >7
1.8-3.0	1.8-3.0, 0-1	1.8-3.0, 1-3	1.8-3.0, 3-7	1.8-3.0, >7
>3.0	>3.0, 0-1	>3.0, 1-3	>3.0, 3-7	>3.0, >7

Sixteen possible combinations of depth and duration are found to categorize a particular flood event. These combinations are given a qualitative nomenclature for easy understanding of the quantifications. While assigning a qualitative name, the ‘first name’ comes from the depth and the ‘last name’ comes from the duration of flood.

Table 4: Qualitative nomenclature of depth and duration of a flood event

Low, Short	Low, Medium	Low, Long	Low, Very Long
Moderate, Short	Moderate, Medium	Moderate, Long	Moderate, Very Long
High, Short	High, Medium	High, Long	High, Very Long
Very High, Short	Very High, Medium	Very High, Long	Very High, Very Long

3.3 Development of Hazard Magnitude

Hazard assessment is done on the basis of flood extent, depth and duration of flood. In this study flood depth is taken as independent parameter and flood duration is considered as dependent parameter. Depth and duration are computed by using the duration determination procedure described earlier. Duration of each flood zone is determined by using the procedure where, four categories of flood depths were used. Water depth of flood falls or rises from the average increase of depth which is determined by using this method. The bowl shape tectonic depression of haor basin helps to develop this method. Besides, it is important to mention that physical observation, information collection from the haor area and model results give a clear idea that flood water rises in the haor basin continuously from the pre-monsoon season. From a combination of flood extent, depth and duration, hazard magnitude is set up in the range of a 0 to 100 scale. Different values of aerial extent and duration based on different depth ranges are used in the following equation to determine the hazard magnitude:

$$HM = \frac{A_r \times d_r}{D_{u,r}} \qquad (4)$$

$D_{u,r} = Duration\ for\ the\ depth\ ranges, d_r$

$A_r = Area\ governed\ in\ the\ depth\ ranges, d_r$

By using equation 4, hazard magnitude is determined for 2year, 10year, 20year and 100year return period flood event. Hazard magnitude specifically represents the discharge of that flood zone. If aerial extent is represented in square meter and depth is in meter the total volume of water passes through the area is obtained. Discharge has been determined from the volume divided by duration which is taken here as an hourly varying quantity. Hazard magnitude is determined by using hazard index defined for each categories of depth and duration of flood event. Finally the hazard magnitude is converted into a traditional scale of 0 to 100.

The maximum depth, extent and duration were selected to represent hazard map for flash flood in pre-monsoon period. Flood inundation extent has been analyzed using the model simulated depth and duration information in Arc-GIS. The inundation map has been reclassified in raster reclassify method, where inundation map is used as input raster. Reclassified map has been converted into polygon feature map using raster to polygon in conversion tools. The entire polygon feature has been dissolved into five categories through generalization technique in data management tools. Area of each category has been calculated in utilities by using 'calculate areas' tools. Hazard maps have been prepared for 2year, 10year, 20year and 100year return period flood events. The generated hazard maps are shown in Figure 3, Figure 4, Figure 5 and Figure 6 respectively.

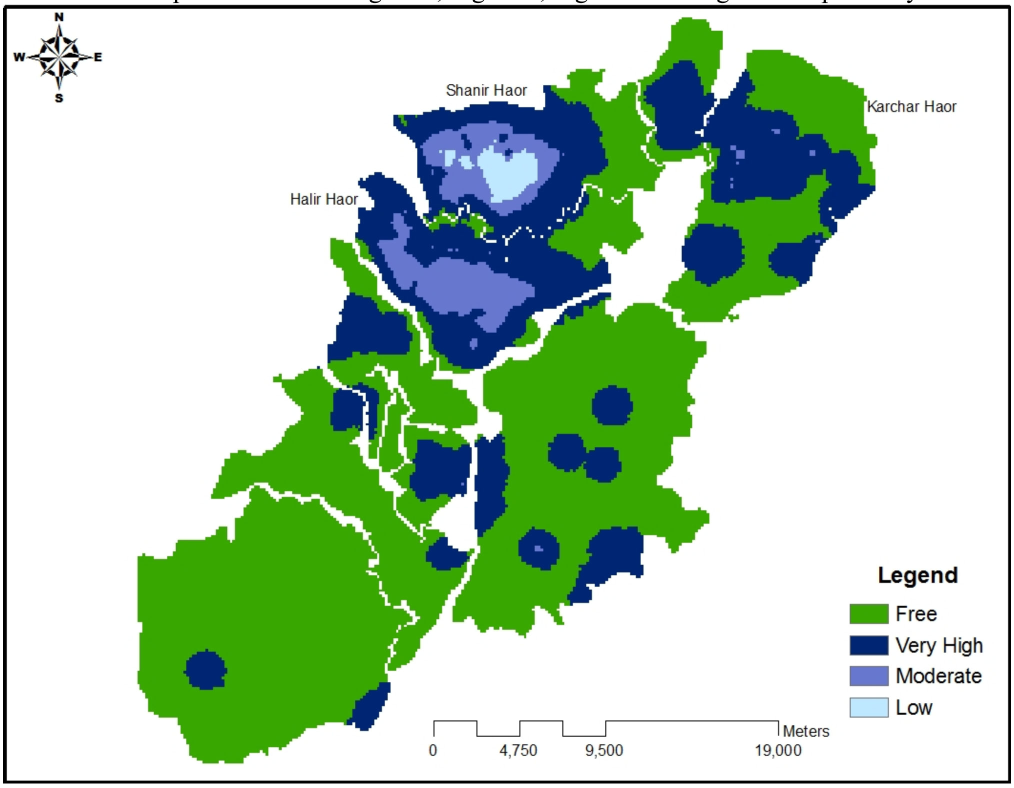

Figure 3: Hazard map for 2year return period flood event

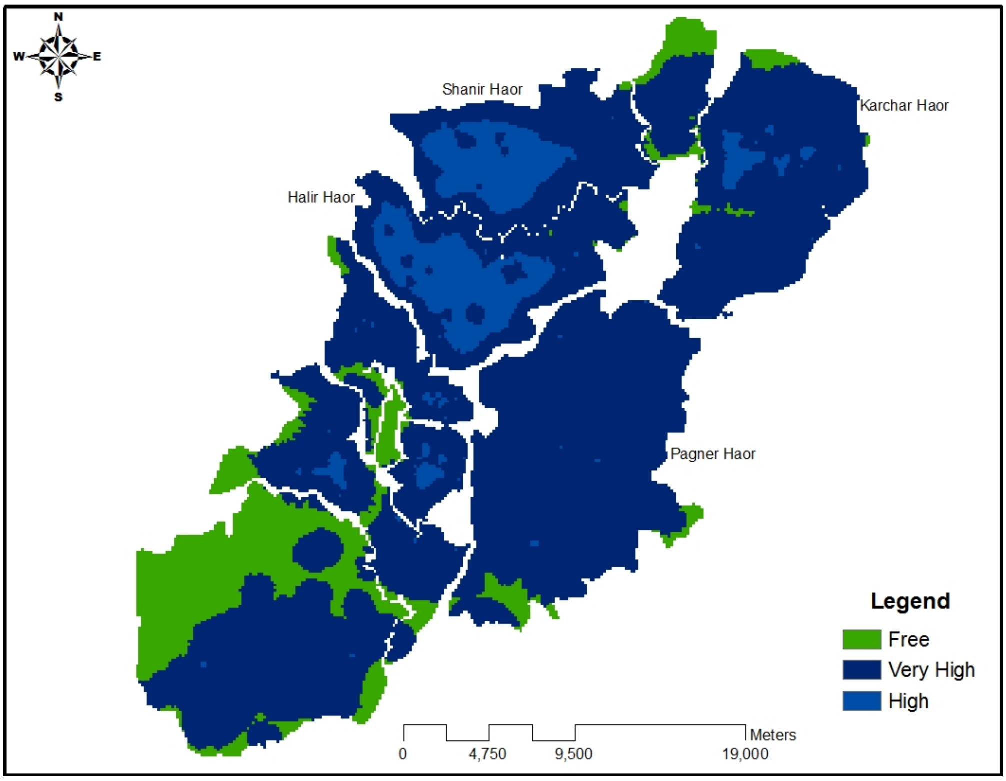

Figure 4: Hazard map for 10year return period flood event

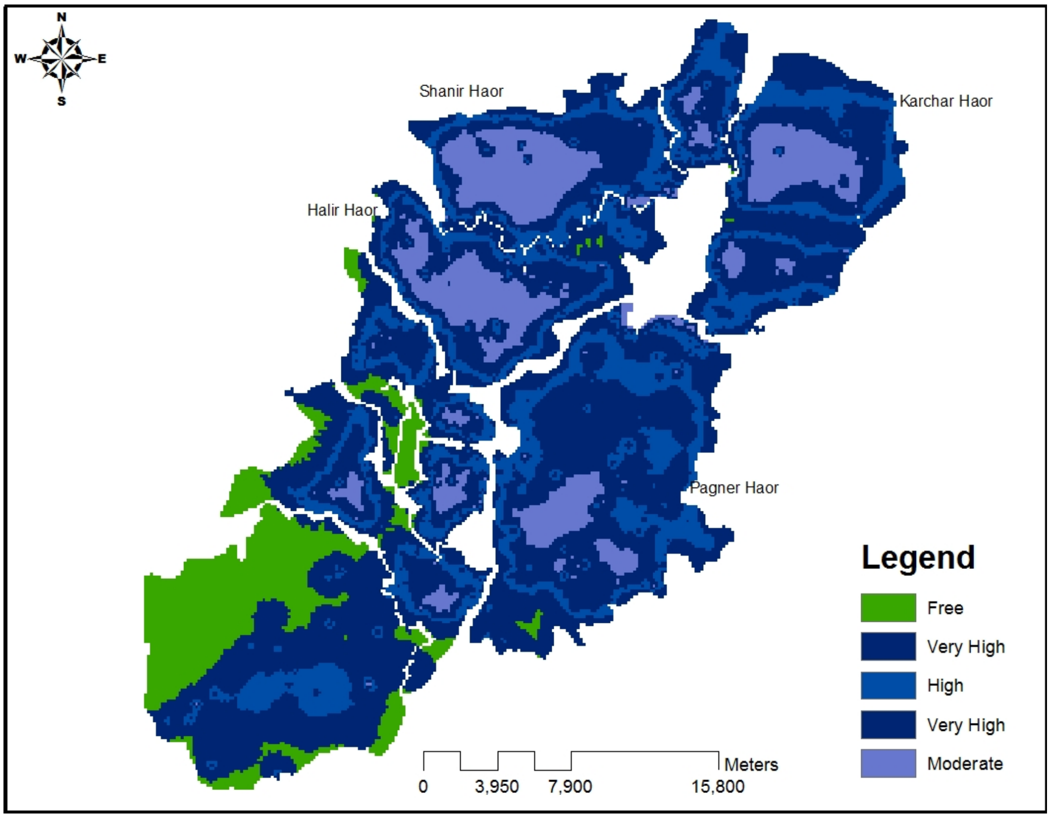

Figure 5: Hazard map for 20year return period flood event

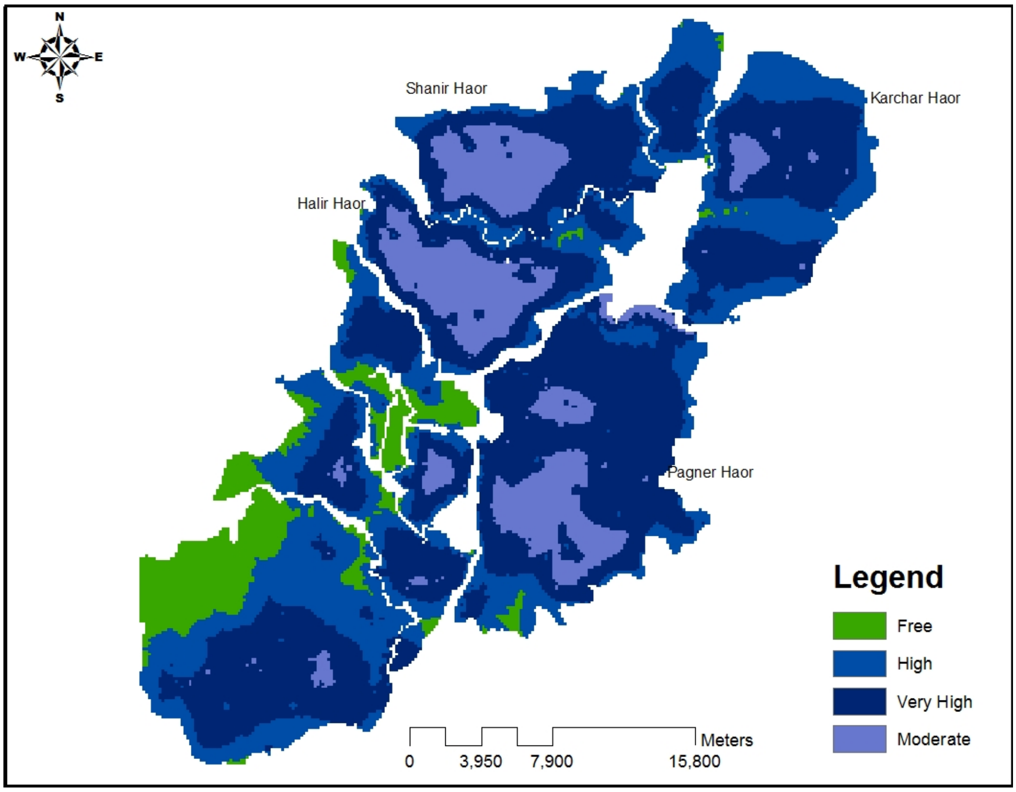

Figure 6: Hazard map for 100year return period flood event

Hazard maps shown in figure 3 to figure 6 has been prepared by adopting the procedure of hazard magnitude computation based on hazard index values of depth and duration. Different hazard zone has different hazard magnitude. For the 2Year return period flood event it has been found that, flooded area at critical date of harvesting is 35.5% of the total area. In the case of 10Year, 20Year and 100Year return period flood event the values of flooded area are 85.6%, 88.7% and 91.2% respectively. For a 2Year return period flood event, 64.5%, 29.5%, 5% and 1% area for the flooded area were found for four ranges of flood depths, namely 0, 0-0.9m, 0.9-1.8m, and 1.8-3.0m respectively. For a 20 Year return period flood event, 11.3%, 30%, 20%, 26% and 12.7% area were found for five ranges of flood depths, namely 0, 0-0.9, 0.9-1.8, 1.8-3.0 and >3.0m respectively. The duration of each flood zone corresponding to each flood depths were found as 1day, 3days, 5days and 10days respectively for 0-0.9m, 0.9-1.8m, 1.8-3.0m and >3.0m flood depths range. For 100 Years return period flood event, 8.8%, 36%, 20%, 26% and 9.2% area were found for five ranges of flood depths 0, 0-0.9, 0.9-1.8, 1.8-3.0 and >3.0m respectively. The duration of each flood zone corresponding each flood depths were found as 2days, 3days, 7days and 21days for 0-0.9m, 0.9-1.8m, 1.8-3.0m and >3.0m flood depth respectively. When duration is considered, fraction of a day is taken as a full day. To compute the hazard magnitude, hazards index for each categories of duration is considered. Hazard magnitudes for different return period of floods and for different ranges of depths are shown in Table 5.

Table 5: Hazard magnitude for different return period flood events

Depth range (m)	Hazard magnitude for different return period flood events			
	2year	10year	20year	100year
0	0	0	0	0
0-0.9	100	100	100	63
0.9-1.8	34	87	66	76
1.8-3.0	8	95	86	100
>3.0	-	69	42	34

From the hazard magnitude table, classification of hazard zone has been identified with the range. Four hazard types were selected on the basis of hazard magnitude range. If the hazard magnitude lies from 0 to 25 then it is represented as low hazard, if HM lies within the values between 25 to 50, 50 to 75 and 75 to 100 then it is represented as moderate, high and very high respectively. For 2Year, 10Year and 20Year return period flood event flood depth ranges 0-0.9m has high aerial extent and higher duration of flood that results very high hazard.

Also hazard magnitude of 95 and 86 for depth class 1.8m-3.0m in the event of 10Year and 20Year flood represent very high hazard. For 100Year return period flood event, zones in the depth classes of 0.9m-1.8m and 1.8m-3.0m are represented as very high hazard.

4. CONCLUSION

During the development of hazard map, hazard is identified by significant color scheme which is commonly used for the purpose of hazard signal. Uses were made for leaf green, dark navy, ultra blue, pacific blue and sodalite blue for representing of hazard free, very high hazard, high hazard, moderate hazard and low hazard respectively. From the comparative analysis of hazard map it has been found that, for a 2year flood, Karchar, Shanir and Halir haor systems have been largely affected than the others. For the flood of 10year, 20year and 100year it is found that all haors in the study area are heavily affected. It is evident that the haor basin is highly risk area for pre-monsoon flash flood. It is usually found that dry season boro crop that the principle cropping pattern in haor basin has largely affected.

ACKNOWLEDGEMENT

Financial support from the MorphoFlood project of the DGIS UNESCO-IHE Programmatic Cooperation supported by the Dutch government to enable the research is hereby acknowledged.

REFERENCES

Alphen, J.V. and Passchier, R. 2007. Atlas of Flood Map, EXCIMAP, WL / Delft Hydraulics Available at: http://sciencesearch.defra.gov.uk/Document.aspx?DocumentID=3646

Chowdhury, J.U. and Karim, M. F. 1996. A risk based zoning of storm surge prone area of the Ganges tidal plain, Journal of Civil Engineering, The Institution of Engineers, Bangladesh, Vol. CE 24, No. 2, 1996

Siddiqui, K.U. and Hossain, A.N. 2006. Options for Flood Risk and Damage Reduction in Bangladesh, Dhaka University Press Ltd

Tingsanchali, T. and Karim, M.F. 2005. Flood hazard and risk analysis in the southwest region of Bangladesh, Hydrol. Process, 19, 2055–2069, John Wiley and Sons Ltd

UNDRO, 1991, Mitigating natural disasters: phenomena, effects and options, A manual for policy makers and planners, United Nations Disaster Relief Organization, New York.

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EROSION TREND ANALYSIS AT CHANDPUR CONFLUENCE OF MEGHNA ESTUARY

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ABSTRACT

Bangladesh is a low lying riverine country formed by a deltaic plain at the confluence of the three mighty rivers Ganges, Brahmaputra, Meghna and their tributaries. Most of the flows of these major rivers are connected to the Bay of Bengal through Meghna Esuary. The Lower Meghna River with a very dynamic estuarine system conveys the maximum flow through the estuary to the Bay of Bengal during dry and wet season (Rahman et al, 2013). The sediment load and flow volume in Lower Meghna is respectively the highest and the third highest (after Amazon and Congo) in the world (Ahmed S. 1998). Owing to this tremendous water volume and sediment load delivered by the rivers, the estuary is an area of an active land erosion-accretion and dynamic water circulation. The main physical processes in the estuary are mainly dependent on the estuarine morpho-dynamics that causes erosion and accretion in the scale of hundred sq. kilometers per year and cause several thousands of people to become landless and homeless every year (Biswajit et al., 2013). Chandpur district situated at Padma and Upper Meghna confluence point in Meghna estuarine system is selected as the study area, since it is the most vulnerable zone for erosion and accretion due to sediment load. In this study, the long term suffering due to erosion, bank shifting characteristics and sand bar formation of the Meghna River has been unveiled. At first a 1D model is set up for an average year to compute the yearly maximum and average discharge of Lower Meghna River at Chadpur confluence. Then the erosion analysis has been done using high resolution of Landsat satellite imagery (Band 4 to Band 7) from the year of 1973 to the year of 2010 and Google Earth image from 2000-2013. Net accretion, net erosion and the erosion trend are derived from the image processed using ILWIS 3.7.2 and ArcMAP 10. It is obvious that the erosion rate was in decreasing order from the year 1973 to 1984, 3.4km2/yr to -8.4km2/yr. Analysis also showed that in the history of the last forty years erosion rate was higher from the year 2003 to 2005, -8.4km2/yr to 15.4km2/yr. Recent year analysis predicted that the deposition is higher than the erosion. However, further research should be focused on future channel shifting pattern, erosion rate and vulnerability of different locations as well as determining effective measures to decrease erosion severity and its consequences.

Keywords: *Erosion, GBM Delta, Confluence, Accretion, Sand Bar*

1. INTRODUCTION

Bangladesh is a riverine country with enriched natural resources. People are directly or indirectly dependent on the rivers of this country. River plays a vital role in the livelihood pattern of the people who are mostly influenced by the rivers. So, morphological characteristics of these rivers are very important issues for the life and livelihood of people.

The most influential single phenomenon to have deep impacts on its culture and economy is the river system. Ironically, these rivers bring along the perennial threat of floods and riverbank erosion. Riverbank erosion is an endemic and recurrent natural hazard in Bangladesh. In a typical year, about 2,400 kilometres of bank line experience major erosion. Millions of people are affected by erosion that destroys standing crops, farmland and homesteads every year. A report of Asian Development Bank (2002) reveals that riverbank erosion displaces more than 100,000 people annually in Bangladesh, resulting in devastating social disparity and poverty.

Bangladesh is a low lying riverine country formed by a deltaic plain at the confluence of the three mighty rivers Ganges, Brahmaputra, Meghna and their tributaries. The Meghna Estuary is a very dynamic estuarine and coastal system. The Lower Meghna is one of the large river systems in the world, which conveys the flow through the estuary to the Bay of Bengal. The sediment load in the Lower Meghna is the highest and the water discharges the third highest (after Amazon and Congo) in the world (Ahmed S. 1998). Owing to this tremendous water and sediment load delivered by the rivers, the estuary is an area of very active land erosion and accretion and dynamic water circulation.

2. STYDY AREA

This study focused mainly at Meghna- Chandpur confluence and tried to establish a trend of erosion and accretion patterns. Erosion trend at this confluence is a very recent concern and few works have been done to investigate the actual scenario behind the causes. The river system at this point is a complex one with large propagation of sand bar and also has a gradual shifting of the whole lower Meghna River in eastward direction.

The study area lies between the latitude 23°25'11.98"N to 23° 2'4.84"N and longitude 90°35'58.81"E to 90°39'15.09"E from the upper Meghna to lower Meghna with a approximate length of 46.5 km. In this paper, from the entire river course only left bank is considered for trend analysis because major erosion had been occurred in the left bank and its adjacent influenced area.

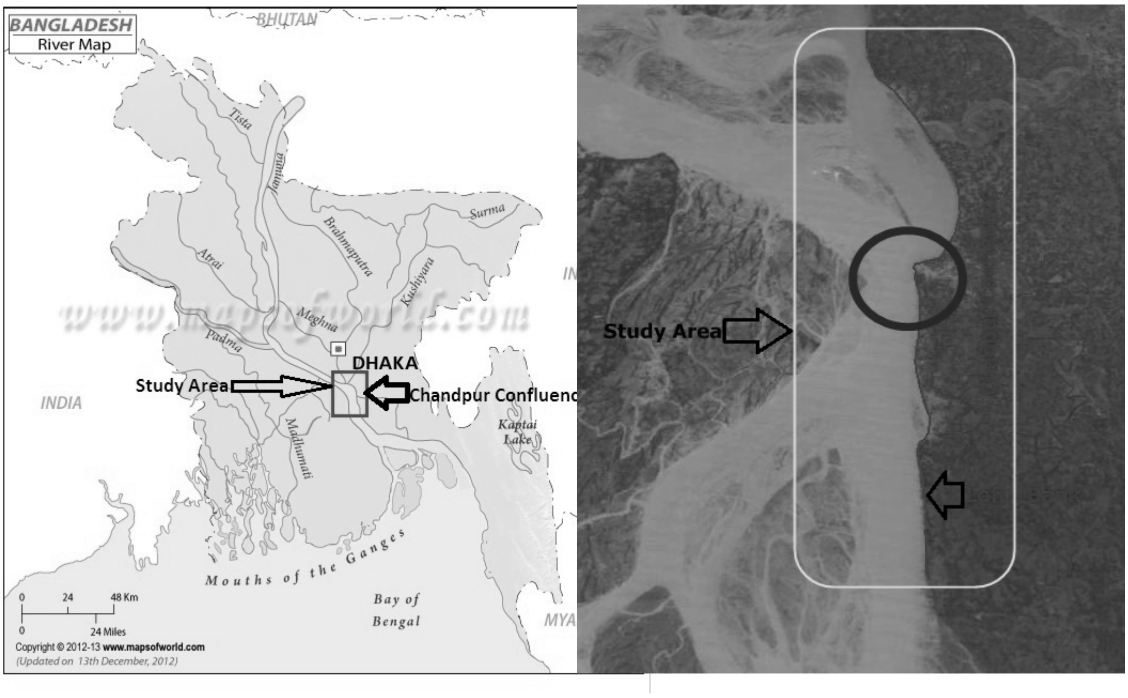

Figure 14: Location of the left bank and study area (Google Earth)

3. MODEL SET-UP AND VALIDATION

As the combined fresh water flow at the confluence is an important parameter that controls the erosion and accretion, it is very important to know the amount of water carried by the complex estuarine system. To compute the maximum discharge, HEC-RAS 1D model has been set up.

HEC-RAS is a one-dimensional steady flow hydraulic model developed by U.S. Army Corps of Engineers, designed to aid hydraulic engineers in channel flow analysis and floodplain determination. HEC-RAS 4.1 version is used in our study to compute discharge at three important rivers, such as Lower Meghna, Arial Khan and Gorai, through which all upstream flows are connected to Bay of Bengal. At the same time these rivers consist the three major estuarine systems named as Eastern Estuarine System, Central Estuarine System and Western estuarine system (Rahman et al, 2013) respectively.

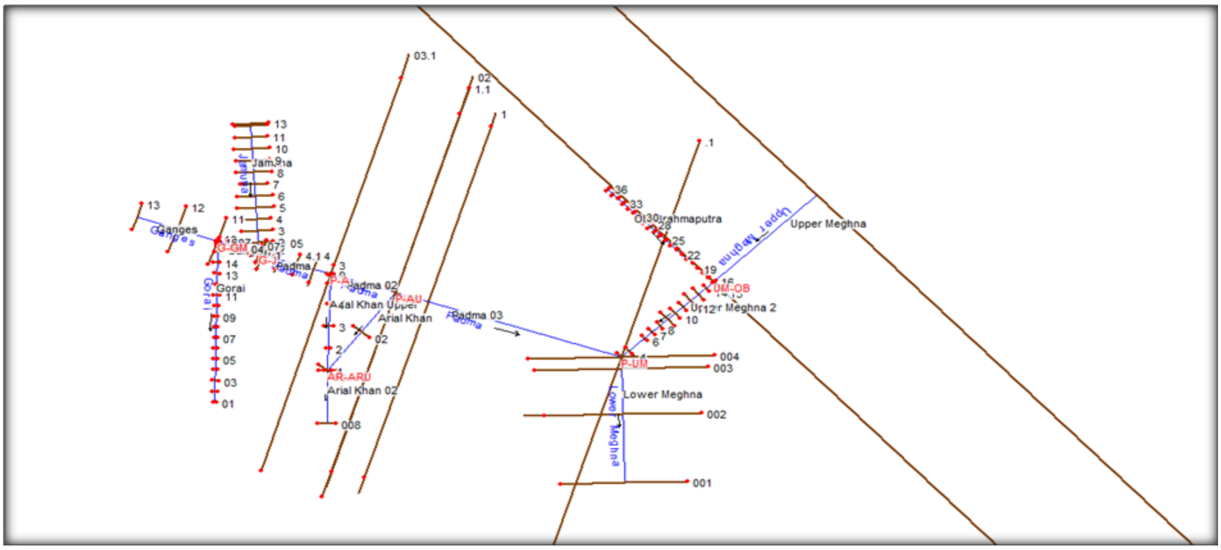

Figure 2: Hec-RAS network to compute discharge at Chadpur Confluence

We chose the year 2000 as an average flood year and prepared the HEC-RAS model for the year.All the major rivers that contribute to the total inflow at Bangladesh has been taken into account in the network. In figure 2 the schematized network is shown. The upstream boundaries in the model were set at Ganges River, Jamuna River and Upper Meghna River where the Lower Meghna, Gorai and Arial Khan River were the downstream rivers up to the reaches where we had required cross-section data.

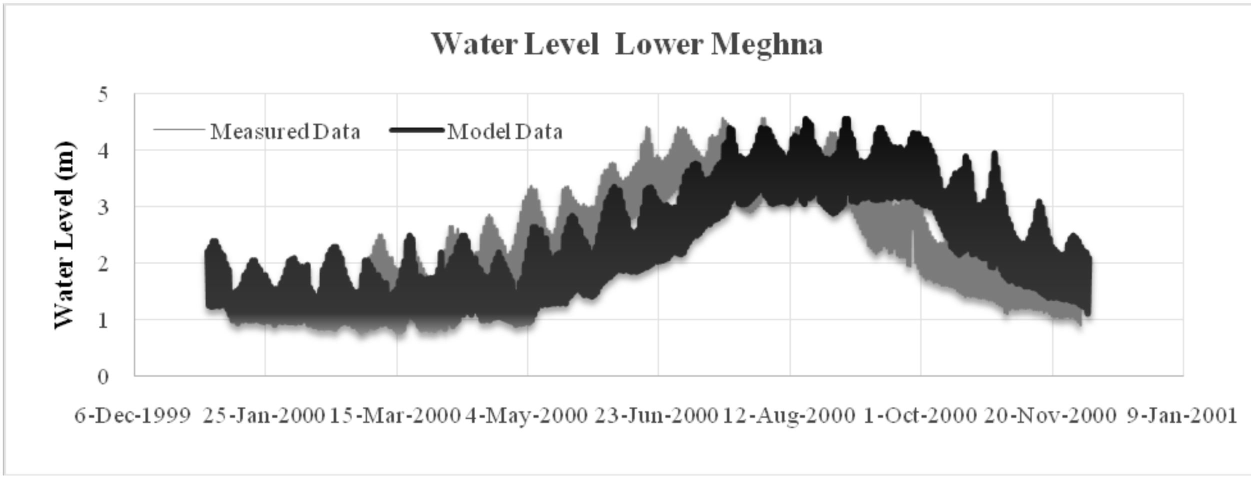

Figure 3: Water level calibration at lower Meghna

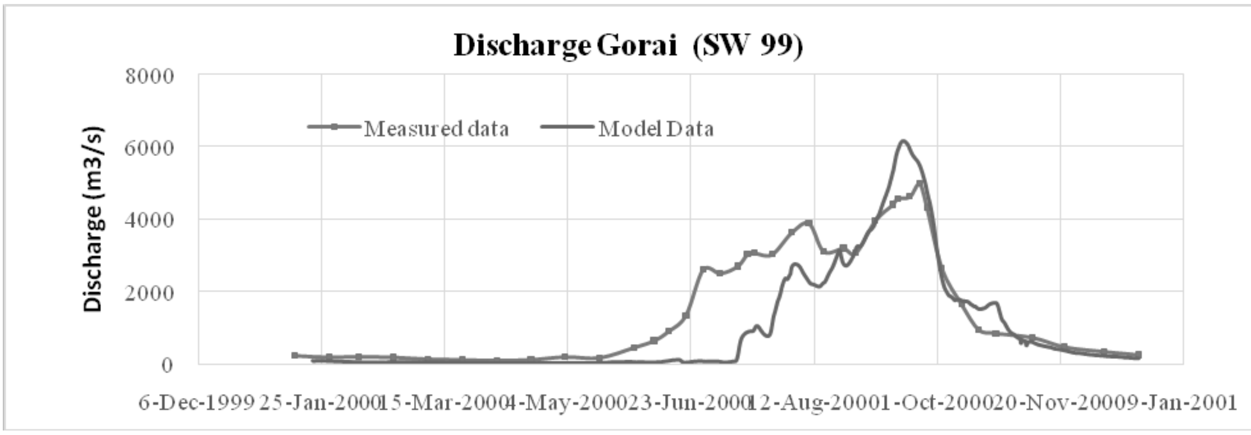

Figure 4: Model discharge comparison with measured discharge at the station SW 99 (Gorai)

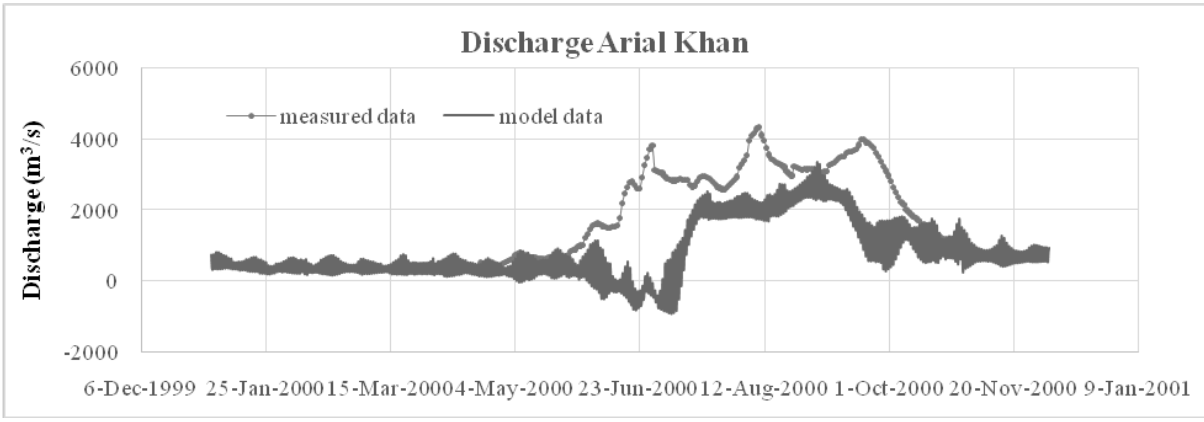

Figure 5: Model discharge comparison with measured discharge at Arial Kha

The simulated outputs are validated with available water level and discharge data from Bangladesh Water Development Board at Lower Meghna, Gorai and Arial Khan River for the year of 2000. The calibrated water level at Lower Meghna River station SW 277.5 is coherent with the observed seasonal and daily tidal range. Moreover, the simulated yearly discharges at Gorai and Arial Khan River followed similar patterns.

4. DATA COLLECTION

To delineate the erosion and accretion pattern, we have used Landsat satellite images of high resolution from Band 4 to Band 7, for the year of 1973 to the year of 2010. Relevant Google Earth images from the year of 2000 to the year of 2013 have also been compared to have better understanding of shifting of left bank at the confluence.

The available satellite images from the year 1973 to the year 2010 have been used at a rudimentary stage to delineate the river bank of the concerned area of interest. By using Google earth pro from the very recent Google Earth historical toolbar necessary Google images are also applied to understand the present scenario of the river bank in comparison with past shifted bank line.

5. IMAGE PROCESSING

Google Earth pro, Geographic Information System (GIS) ArchMap 10 and ILWIS 3.7.2 are the tools used extensively for the image processing and understanding the shifting patterns of the left bank. As the years for our study have been selected according to the availability of data human judgments were applied while processing the available images and the demarcation of shoreline shifting.

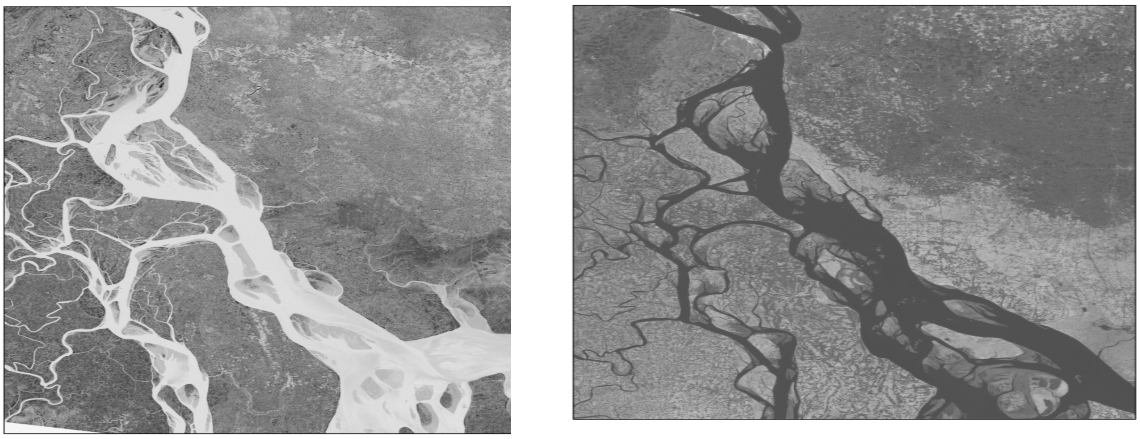

Figure 6: Landsat images of 1973 and 2010 respectively

6. RESULTS

6.1 Discharge Computation

From the flow hydrographs in figure: 6, 7, 8, it is clearly visible that Meghna estuary carries the maximum flow over other estuarine systems. The maximum discharge at Lower MEGHNA River at Chadpur confluence occurred during monsoon on August which is about 94,089 m3/s. At Gorai and Arial Khan River it was found approximately 22100 m3/s and 1100 m3/s respectively. It can be said that during monsoon the flow volume at Lower Meghna River is more than five times of the total flow of the other two rivers which carries the water to further downstream. On the other hand, during dry season almost 90% flow of the total flow of these three rivers are carried by Lower Meghna River. So it obvious that the Chadpur confluence at Lower Meghan River transports the maximum flow towards the downstream regions of Bangladesh. This huge flow and its dynamics are responsible for the erosion –accretion patterns, sediment transport and bank shifting of the river Meghna.

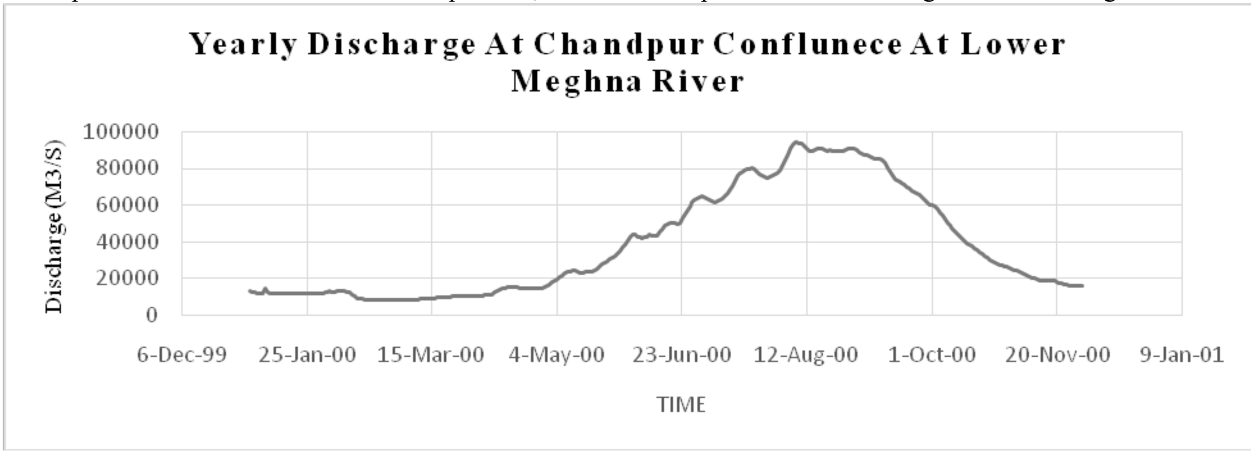

Figure 7: Computed yearly discharge variation over a year at Lower Meghna River

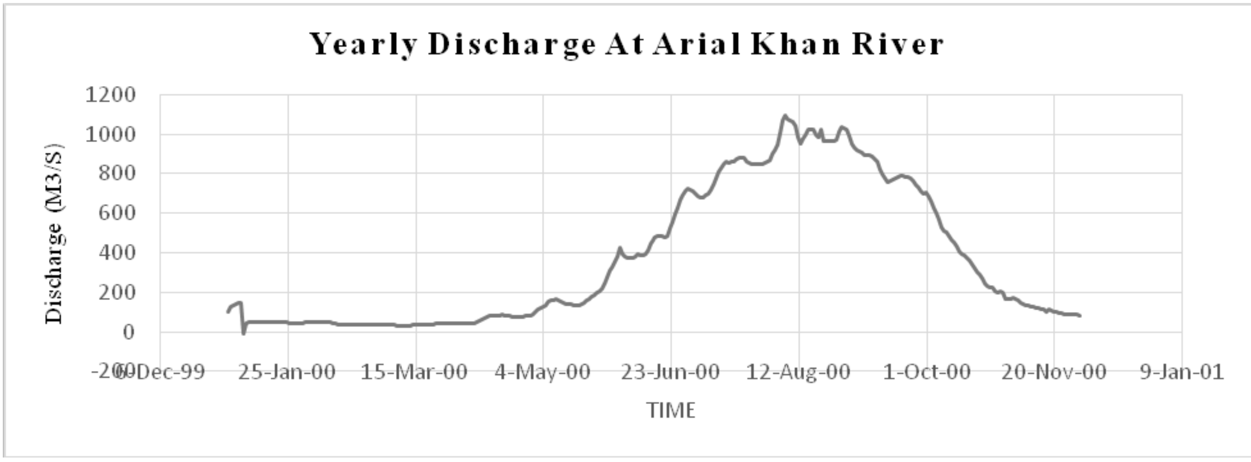

Figure 8: Computed yearly discharge variation over a year at Arial Khan River

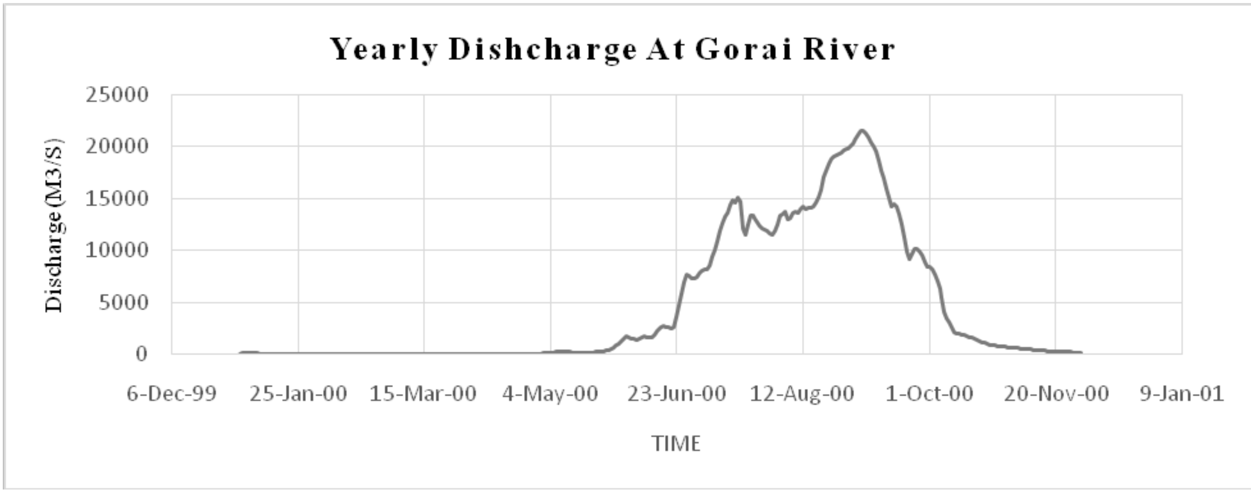

Figure 9: Computed yearly discharge variation over a year at Gorai River

6.2 Accreted and Eroded Area

The combined fresh water flow and sediments coming through the lower Meghna River are distributed into Meghna Estuary though the Shabazpur, Hatia and Sandwip channels. These areas comprises of many channels and islands of different size and shape. Due to the very dynamic nature of these channels and islands the planform of the Meghna Estuary is always changing and very difficult to predict the nature.

The magnitude of erosion and accretion in Meghna Estuary is represented in the following table from 1973 to 2010 and a figure has been established to understand the trend. The table shows the amount of area accreted and eroded for the different time periods. From the figure, it can be deducted that the area of accretion was higher at the 1994-1996 period and followed a downward trend up to the period of 2005-2008 from 40.6 km^2 to only 6.8 km^2. From the period of 2005-2008 to the period of 2008-2010 a sudden rise is noticeable. On the other hand, in case of erosion there was also a downward trend immediately observed from the period of 1973-1984 to the period of 2001-2003 from 53.1 km^2 to 8.7 km^2 and then again raised at 40.5 km^2 in the period of 2003-2005. The area of erosion was decreasing from the period of 2003-2005 to the period of 2008-2010 from 40.5km^2 to 9.7 km^2.

Table 1: Accreted and eroded area for the different time span at the chandpur confluence

Year	Accreted Area (km^2)	Eroded Area (km^2)
1973-1984	16	53.1
1984-1996	40.6	45
1996-2001	25.4	22.2
2001-2003	25.5	8.7
2003-2005	9.6	40.5
2005-2008	6.8	15.4
2008-2010	21.1	9.7

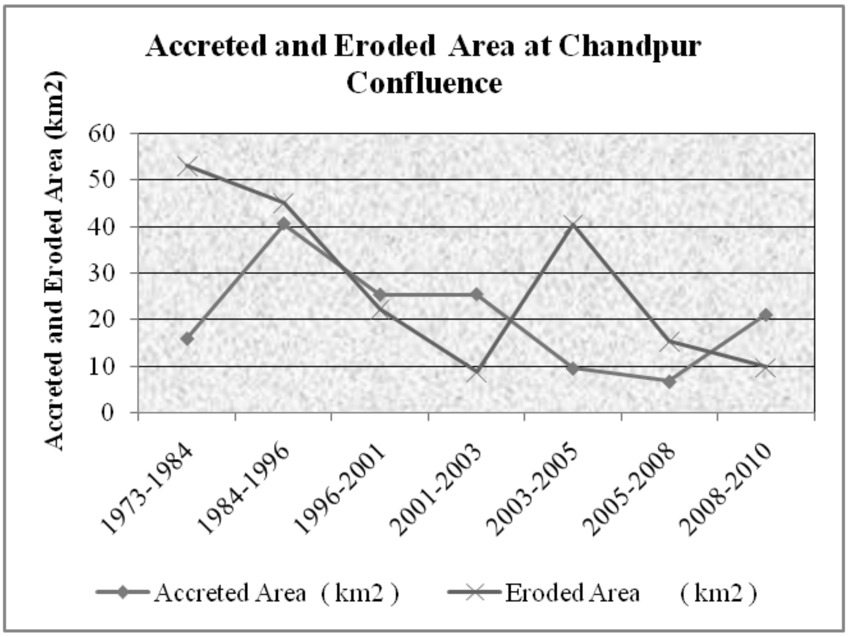

Figure 10: Amount of area of deposition and erosion

6.3 Net Erosion and Erosion Rate

Net erosion and erosion rate was observed along the shoreline of chandpur district. It is immediately apparent that erosion is a dominant process at the chandpur confluence. From the table and figure listed below it can be found that accretion was dominant for the time period of 1973-1984 and then followed a downward inclination upto the time period of 2001-2003. Although, at the time period 2003-2005 erosion rate was jumped abruptly from -8.4 km^2/yr to 15.4 km^2/yr indicating higher accretion, still it was approaching towards a downward direction in the recent years.

Table 2: Net erosion rate for the different time span at the chandpur confluence

Year	Net Erosion (km^2)	Rate of Erosion (km^2/ yr.)
1973-1984	37.1	3.4
1984-1996	4.4	0.4
1996-2001	-3.2	-0.6
2001-2003	-16.8	-8.4
2003-2005	30.9	15.4
2005-2008	8.5	2.8
2008-2010	-11.4	-5.7

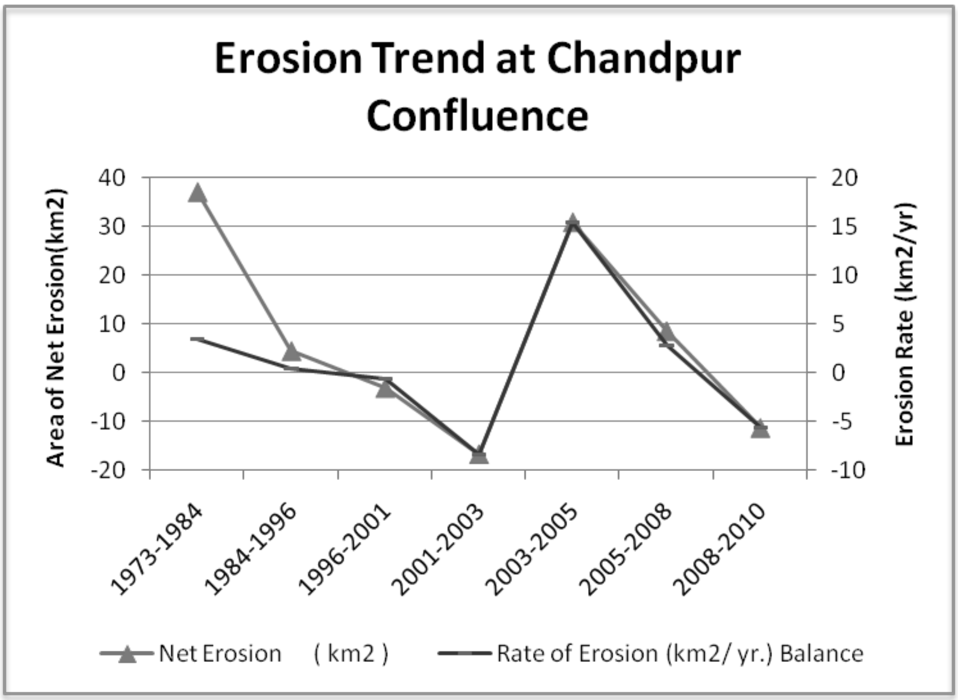

Figure 10: Erosion and erosion rate trend at the confluence

6.4 Land Shifting

Land shifting is an important and governing parameter for the morphological changes of the river bank. We have shown two shapefiles of the shorelines for the year of 1973 and 2010 and from which total bank shifting was determined with the help of GIS application. From the confluence (90.639 E and 23.23 N) total shifting was 1118 m at the upper Meghna (90.646 E and 23.303 N) and 872 m at the lower Meghna (90.639 E and 23189 N).

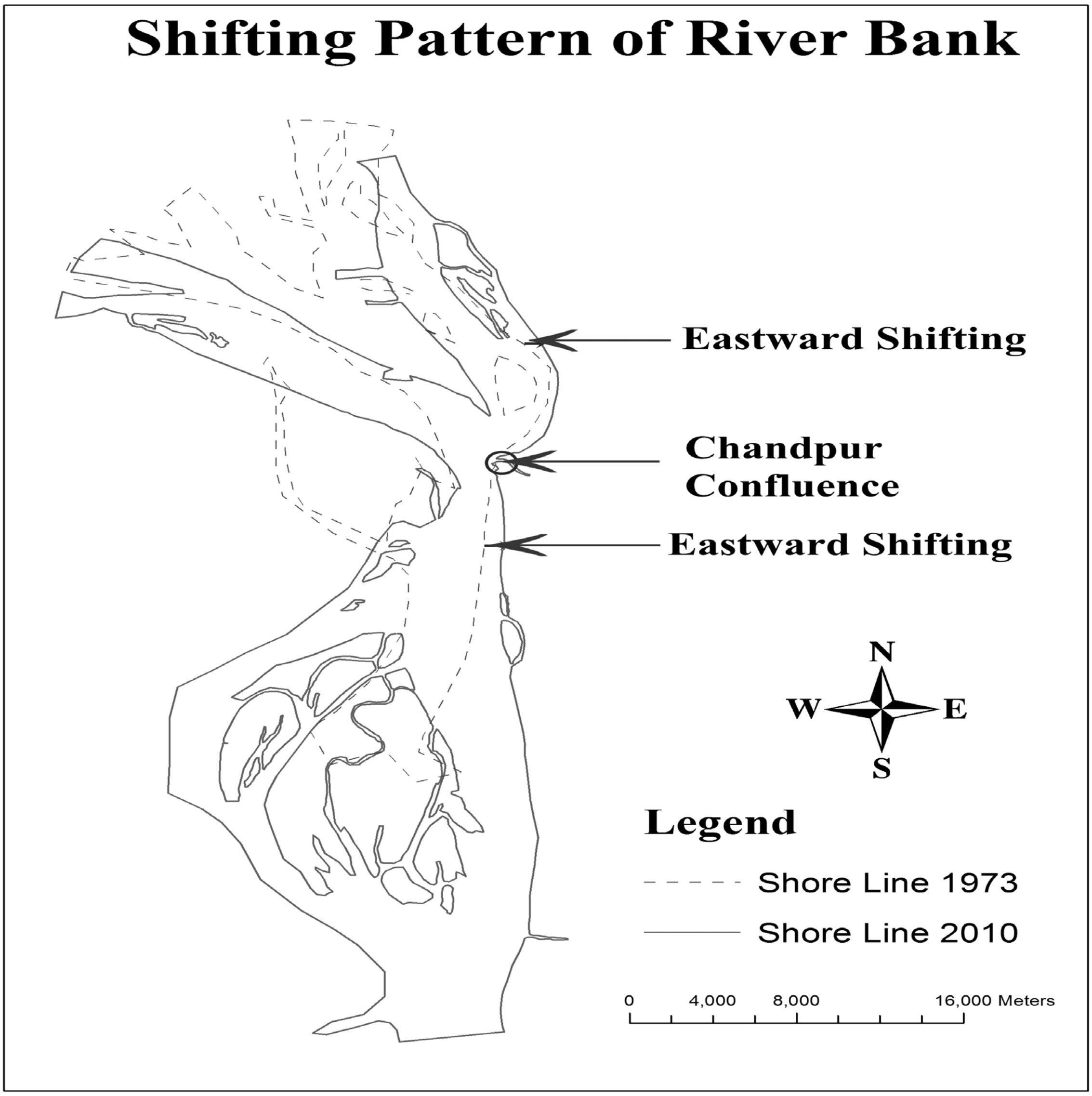

Figure 11: Bank line shifting for the year of 1973 and 2010

7. CONCLUSIONS

HEC-RAS 1D model,some aerial photographs for different years and numerical analysis have been used to find out the discharge, length of bank shifting and the erosion trend of chandpur confluence. It is found that the erosion rate was in decreasing order from the year 1973 to 1984. Analysis also showed that in the history of the last forty years erosion rate was higher from the year 2003 to 2005. Recent year analysis predicted that the deposition is higher than the erosion. The river course at this point shows a bank shifting north and south-eastward. Tidal flow, combined flow of Padma and Meghna and the bank shifting are considered as probable main reasons for the erosion at the chandpur confluence. Since chandpur district has a direct relation of livelihood with this erosion, special measures should be taken into account. Further research should be focused on future channel shifting pattern, erosion rate and vulnerability of different locations as well as determining effective measures to decrease erosion severity and its negative consequences.

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REFERENCES

Haque, C.E., and Hossain, Z. (1988). "Riverbank Erosion in Bangladesh", Geographical Review, Vol. 78(1), pp. 20-31.

Julian. J.P., and Torres, R. (2006). "Hydraulic Erosion of Cohesive Riverbanks", Geomorphology, Vol. 76, pp. 193–206.

Chowdhury, R.K., Maruf, B.U. and Chowdhury, A. I. (2007). "Climate change would intensify river erosion in Bangladesh, Impact of climate change in Bangladesh", Dhaka.

Ahmed, N. (1989). "Study of some bank protective works in Bangladesh", Unpublished M.Sc. thesis, Dept. of water resource engineering, BUET, Dhaka.

Nath, B., Naznin, S.N. and Alak, P. (2013). "Trends analysis of river bank erosion at Chandpur, Bangladesh: A remote sensing and GIS approach", INTERNATIONAL JOURNAL OF GEOMATICS AND GEOSCIENCES Vol. 3(3).

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APPLICATION OF GIS IN RISK ASSESSMENT OF STORM SURGE IN BHOLA DISTRICT

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ABSTRACT

Due to the funnel shaped coastline of Bangladesh, the coastal regions are highly susceptible to these cyclones. Bhola, an offshore island situated at mid-southern coastal zone of Bangladesh, has been subjected to many storm surge events throughout its history due to its exposure to the Bay of Bengal. Risk level is the probability of occurrences of damages. The depth and the extent of storm surge flooding for 50 years return period has been determined. It has been termed as the hazard index. The study area is divided into five categories based on their natural and administrative features. Different features of different economic values along with population density have been considered in terms of vulnerability index. Finally utilizing both the hazard index and the vulnerability index, risk index for each upazila in the study area has been determined. On the basis of this risk assessment, the study area is classified into four categories: the low risk area, the moderate risk area, the high risk area, and the severe risk area. GIS technology has been used to perform the risk assessment which reveals that some regions of the study area. Bhola District falls under severe risk zone for the storm surge height, whereas a significant percentage of the area are in high risk and in some regions severe risk zones.

Keywords: *Storm Surge, Damage, GIS, Risk Assessment, Coastal Region*

1. INTRODUCTION

Bangladesh, due to its unique geographic location, suffers from devastating tropical cyclones frequently. When cyclones make landfall the funnel-shaped northern portion of the Bay of Bengal causes tidal bores, and thousands of people living in the coastal areas are affected. Some of the most devastating natural disasters in recorded history with high casualties were tropical cyclones that hit the region now forming Bangladesh. Among them, the 1970 Bhola cyclone alone claimed more than 500,000 lives (Ouderm, 2007). The Bhola district, situated in the Bay of Bengal, has been selected for the study. It is a densely populated island (456/km^2 respectively) and is likely to be affected by storm surges and cyclones (BBS, 2001). The objectives of the present study are selected with a view to determine the extent of damages caused by storm surges and cyclones in the island and to recommend further actions to safeguard people and assets. The objectives include:

1. To determine the amount of vulnerable features to storm surges for Bhola island.
2. To determine the risk level of each union.
3. To propose modification of land use pattern based on the risk level.

The aim of the study is assessment of storm surge risk Bhola in the mid-southern coastal zone of Bangladesh. In order to achieve the above stated objectives, existing storm surge risk assessment techniques have been utilized from the literature review. In order to provide a risk assessment technique for the study area a suitable method is proposed. The planners and decision makers will be able to use this representation of risk to prioritize future development work and to protect the existing resources in the study area.

2. METHODOLOGY

2.1 Study Area

The study area, Bhola District (Barisal division), is an offshore island with an area of 3403.48 square km. The district now consists of 7 upazilas, 60 union parishads, 409 mouzas, 460 villages, 5 municipalities, 45 wards and 62 mahallas. The upazilas are Bhola Sadar, Daulatkhan, Burhanuddin, Tazumuddin, Manpura, Lalmohan and Charfasson. From the demographic data of Bhola, it is found that most of the people live on agricultural activities (about 60 percent). Fishing is also one of the major occupations in the island where about 5.9 percent people live on it (Banglapedia, 2006). Social infrastructures in the island include 540 educational institutions (12 kindergarten and 264 secondary schools, 7 colleges and 257 madrasas), common facilities like police stations, banks, post offices, 1 hospital and 7 upazila health complexes for each of the upazilas and family welfare centers etc. (Cultural Survey Report of Bhola, 2007).

Fig. 2.1: 22.6903°N 90.6525°E aerial view of Bhola (Google Maps)

2.2 Collection of Data

According to Intergovernmental Oceanographic Commission (IOC, 2009), the process of risk assessment involves few steps for determining hazard risk for a community. These steps are the assessment of hazard, assessment of vulnerability, assessment of community preparedness and finally the assessment of risk. The process can be described by a simple diagram (Figure 2.2).

Figure 2.2: The Risk Assessment Process Described by IOC

The process of collecting the data and sources of the existing inventories are described as following:

2.3 Development of Elevation Maps

Digital Elevation Model (DEM) of the coastal region of Bangladesh (i.e. land topography) and the bathymetric data of the rivers and estuaries as well as the Bay of Bengal have used for developing the model. Main source of the DEM data is the FINNMAP land survey (finnmap.com). The model has been modified in the rivers of the study area with the recent data of 2009 surveyed under another project of IWM.

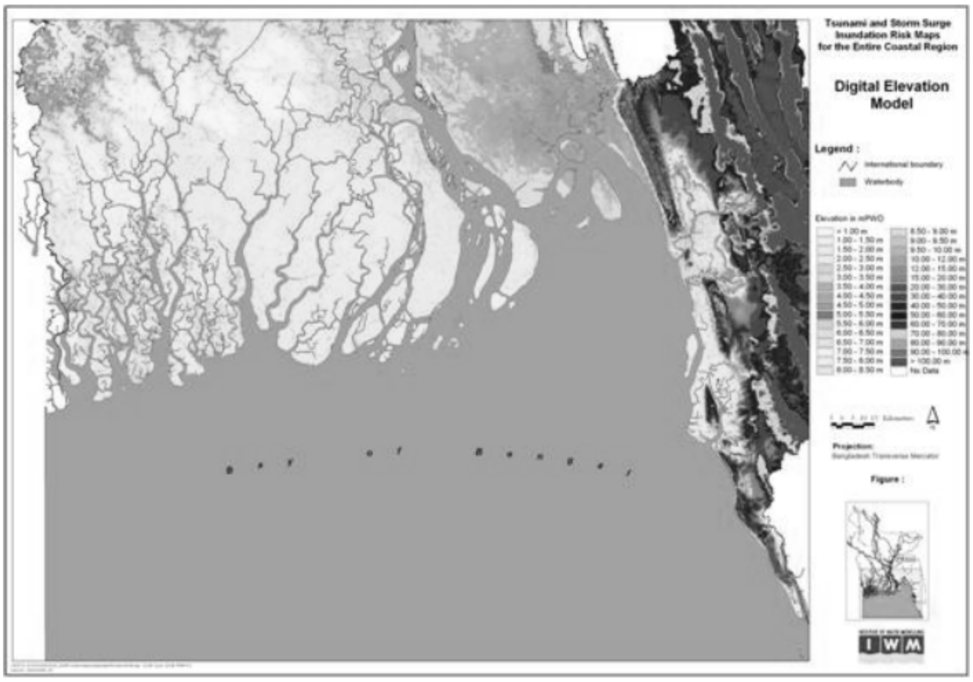

Fig 2.3: Digital Elevation Model of Coastal Region of Bangladesh (IWM)

2.4 Storm Surge Height Data

In this study, storm surge height data has been collected from IWM office for 50 year return period. This height is based on the simulation results of severe cyclone (1960 – 2009).

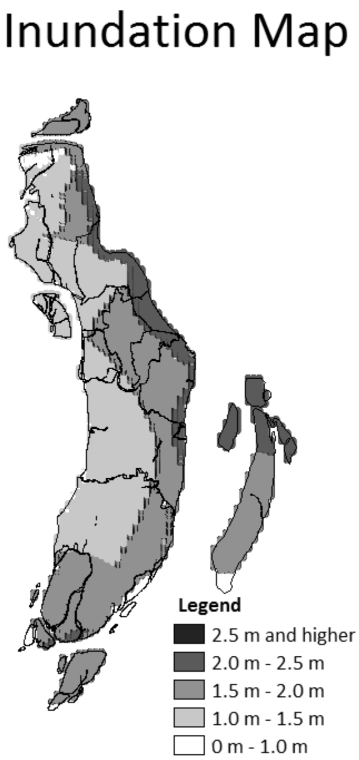

Fig 2.4: Storm Surge potential Inundation map for 50 years of return period

2.5 Risk Assessment and Mapping

According to Abdullah and Hoque, M.M. (1995) the risk of a storm surge for a particular area depends mainly upon depth of inundation, population densities and land use. The method of assessment of risk from such disaster is expressed by the risk index.

Risk Index, RI = HF × VF (2.1)

Where, HF = Hazard Factor

VF = Vulnerability Factor

$$HF = \frac{10 * Hazard\ Index\ of\ an\ Area}{Highest\ Hazard\ Index} \quad (2.2)$$

$$VF = \frac{10 * Importance\ Index\ of\ an\ Area}{Highest\ Importance\ Index} \quad (2.3)$$

RI values for different area have been calculated with the above eqution. Finally RI values are mapped with the help of ArcView GIS 3.3.

2.6 Preparation of Inundation Map / Hazard Map

The hazard index is defined as the element that causes the risk and in this case the depth of the storm surge is the hazard index. The data is laid on the map with the help of GIS tools. The depth of inundation is taken as the hazard index for calculating the hazard factor (HF). The hazard index is scaled from 1 to 5.

Table 2.1: Definition of Hazard Index

Depth of Inundation (m)	Hazard Index
> 2.5	5
2.0 – 2.5	4
1.5 – 2.0	3
1.0 – 1.5	2
0 – 1.5	1

The hazard factors are calculated by using equation (2.2) and then finally the risk index is calculated by the equation (2.1). The result from the above calculation and the risk index for different area for different return periods are determined (Abdullah & Hoque, 1995).

2.7 Preparation of Vulnerability Map

The importance index is defined as the element that indicates the economic value of the areas likely to be flooded and in this case the land use type is the importance index. On the basis of the importance of land use, the study area divided into five zones A, B, C, D and E. The tourism industrial area belongs to A, commercial fishery sector and whole sale market belong to B, urban areas belong to C, unplanned rural settlements belong to D and others including agricultural lands belong to E. The areas under A and B get the highest importance index as 5 score and areas under E get the lowest importance as 1 score. Areas occupied by C and D are given as 3 and 4 scores as importance index respectively.

The importance index is defined as the element that indicates the economic value of the area flooded and in this case the land-use type is the importance index. In this method the higher value of the risk index the higher the degree of risk.

2.8 Risk Classification and Risk Mapping

On the basis of the above findings, the areas are classified into following four categories:

- Low risk area with risk index value $1 \leq RI \leq 10$;
- Moderate risk area with risk index value $11 \leq RI \leq 20$;
- Higher risk area with risk index value $21 \leq R \leq 30$; and
- Severe risk index value $31 \leq RI \leq 50$.

2.9 Analysis and Presentation of Data Using GIS Tools

Disaster risk information is spatial in nature; hence Geographic Information System (GIS) plays a vital role in any disaster risk assessment study. A Geographic Information System (GIS) is a technological tool for comprehending geography and making intelligent decisions. There are reasons to believe that the utility of Geographic Information System (GIS) for natural hazard risk and disaster management will expand as spatial databases become more widely available, the cost of software decreases and as risk manager acquire GIS expertise. It is also likely that GIS use will extend beyond mapping, towards a richer use of its spatial analytical capabilities. Invariably risk managers will also demand access to decision support tools that allow them to manage and understand the complicated nature of disaster. It is likely that GIS-based risk and disaster management will become a feature state and local government's natural hazard risk management procedures. In addition to a range of other disaster risk management objectives such as testing vehicle response time, communication infrastructure and evaluating disaster plans, the scenario aimed to assess the utility of GIS for real-time risk decision making. Results from scenario observations and post-scenario interviews with risk manager highlight the limitations of this technology for real-time applications and disaster management in particular. The research, however, shows that real-time disaster applications of GIS have very specific requirements which are significantly different from long-term decision making for disaster planning. These requirements are examined in detail as the lessons learnt may be valuable for other disaster management planning utilizing GIS.

The computed storm surge results for the case study have been mapped within a GIS, which has been used to evaluate infrastructure and population areas at risk. Once storm surge heights have been computed for the long-term (50 year return period) database, these must then be subjected to a suitable spatial analysis. From this analysis it will be possible to estimate the probable risky areas for inundation.

The visual presentation of the storm surge height data has been presented in this thesis and the following steps are followed to prepare an inundation map:

- First, a base map has been prepared on which all generated data can be superimposed. This mapping procedure is conducted by digitizing (on screen) an existing image of the study area with proper geo referencing.
- The map should contain topographic information in the form of northing/easting or spot elevations, or both.
- Then storm surge height data at various points are converted to hazard index and corresponding hazard index values are assigned to relevant grids.
- Hazard index is represented in graduated color.

3. RESULTS OF ASSESSMENT OF HAZARD

Using the GIS tool, the assessment of hazard in the study area was performed. The key objective of the hazard assessment was to assess the geographical extent of the inundation pattern in the island. Each range of inundation in the area was digitized and grouped into various categories. Figure 3.1 shows the inundation map.

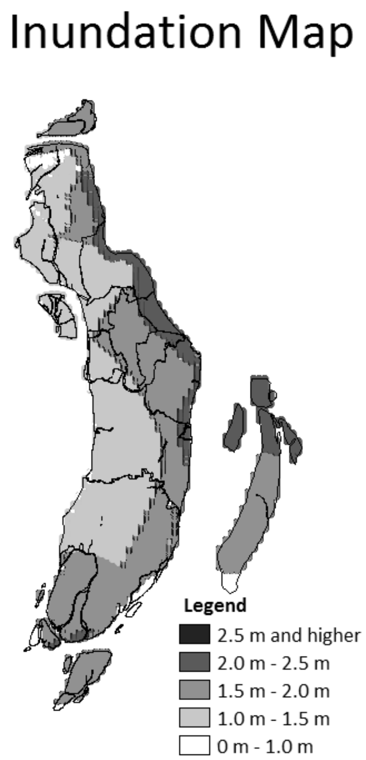

Fig 3.1: Bhola Inundation map for 50 years of return period

From the GIS analysis (digitization, intersection and calculation of area) of the inundation map Table 3.1 represents the area inundated in different upazilas of the district Figure 3.2 demonstrates comparison of total area and inundated area for each union. Charfasson is the largest union in Bhola and the extent of inundation in terms of area in this upazila is not much in comparison with the other upazilas. From the analysis it is also seen that, about 133.04 square kilometers of area will be submerged by more than 2 meter of water in Manpura Island alone. It is also seen that 9.29% of the whole district will be submerged below 2 meter of water and 47.92% will be inundated by a range of 1.5 meter to 2.0 meter.

Table 3.1: Inundation Table for 7 Upazilas of Bhola

Upazila	Land Area (sq. km)	Inundation Area (sq. km)			
		2.0 m and above	1.5 m – 2.0 m	1.0 m – 1.5 m	Below 1.0 m
Bhola Sadar	575.82	40.91	165.40	337.93	31.58
Daulatkhan	197.70	64.00	4.59	127.84	1.28
Burhanuddin	486.81	6.28	239.74	184.30	56.49
Tazumuddin	201.19	49.73	151.46	0	0
Manpura	368.91	133.04	235.88	0	0
Lalmohan	523.38	22.46	128.12	372.806	0
Charfasson	1059.33	0	705.73	353.59	0
Total	**3403.48**	**316.43**	**1630.93**	**1376.45**	**79.68**

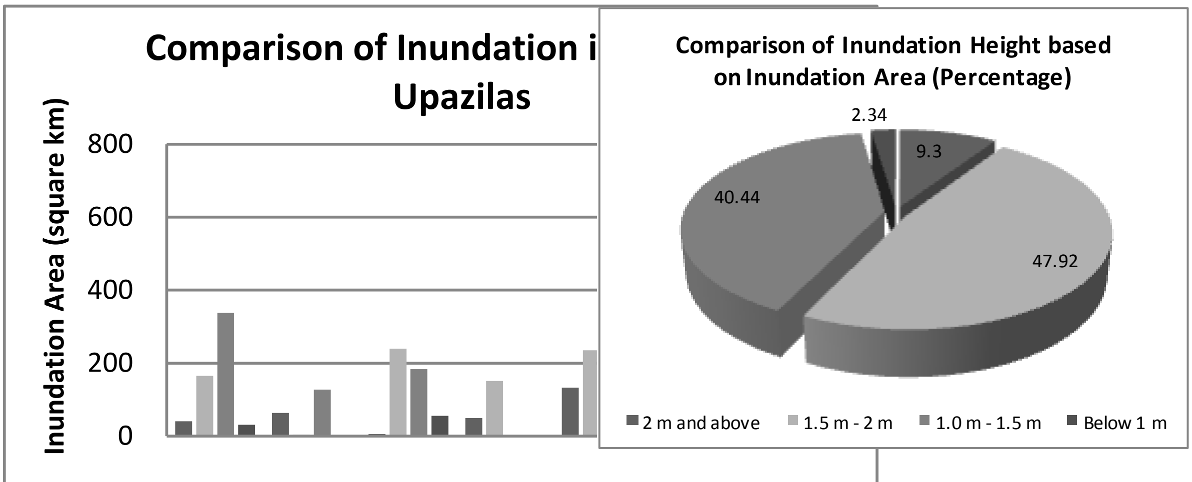

Fig 3.2 & 3.3: Comparison of Inundation in Different Upazilas in terms of Area and Comparison of Inundation Height based on Inundation Area

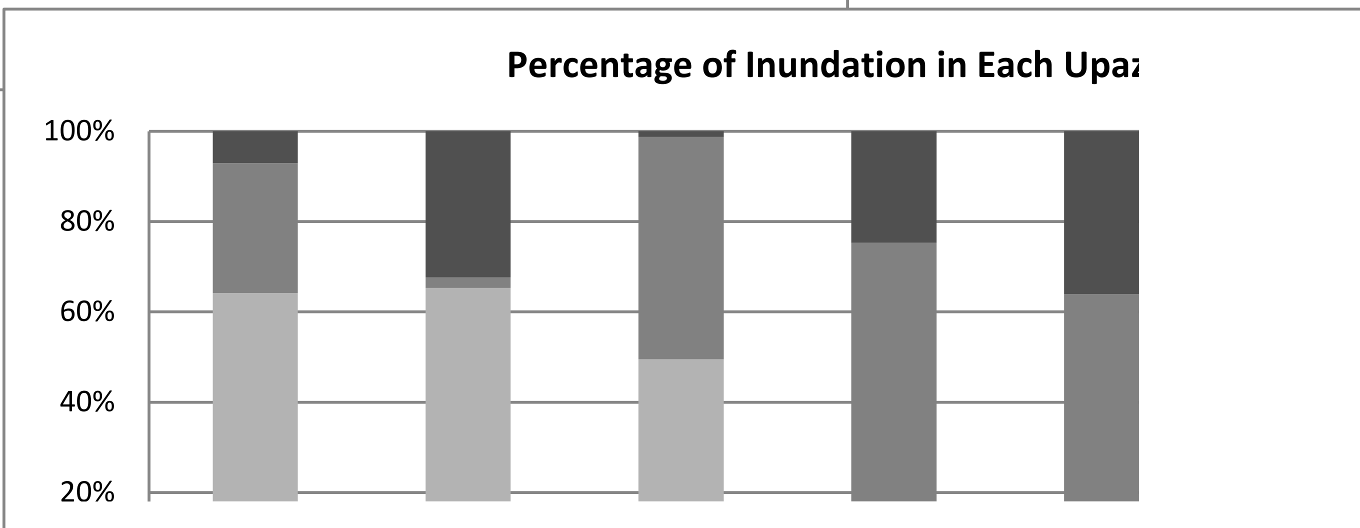

Fig. 3.4: Percentage of Area Inundation in Each Upazila of Bhola

From Fig 3.4 it can be seen that more than 50% of upazilas Burhanuddin, Tazmuddin, Manpura and Charfassion are subjected to more than 1.5 m inundation.

3.1 Vulnerability Map

A vulnerability map has been developed with the help of importance of the respective area. The analysis considers exposure assessment as well as importance analysis. The final vulnerability map has been shown in figure, which will be utilized to generate the ultimate storm surge risk map for the study area.

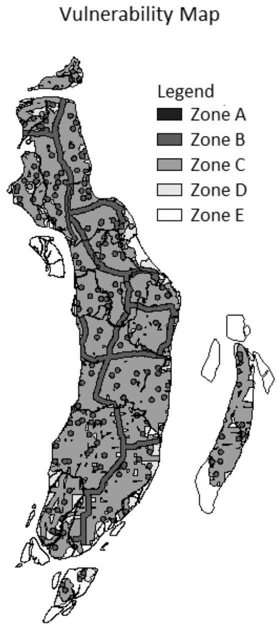

Fig. 3.5: Storm Surge Vulnerability Map

3.2 Results of Risk Assessment

Using the GIS tool, the risk assessment was also performed which is the main objective of the thesis. The final assessment map has been shown in Figure 3.7. The risk index has been calculated multiplying the hazard index with the vulnerability index.

Table 3.2: Risk Index Table for 7 Upazilas of Bhola

Upazila	**Land Area (sq. km)**	**Zones (sq. km)**			
		Severe Risk (31 < RI < 50)	High Risk (30 < RI < 21)	Moderate (20 < RI < 11)	Low Risk (10 < RI < 1)
Bhola Sadar	575.82	40.91	65.4	332.61	136.9
Daulatkhan	197.7	33.75	47.43	116.42	0.1
Burhanuddin	486.81	11.55	92.9	380.39	1.16
Tazumuddin	201.19	18.93	62.86	113.93	5.43
Manpura	368.91	4.14	46.15	198.79	119.83
Lalmohan	523.38	2.44	36.01	478.2	6.73
Charfasson	1059.33	0	98.78	899.21	61.33
Total	**3413.14**	**111.72**	**449.53**	**2519.55**	**331.48**

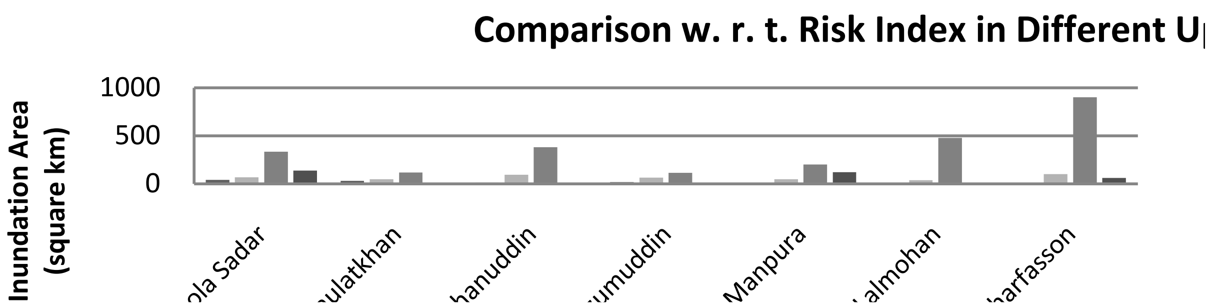

Fig 3.6: Comparison with respect to Risk Index in Different Upazilas in terms of Area

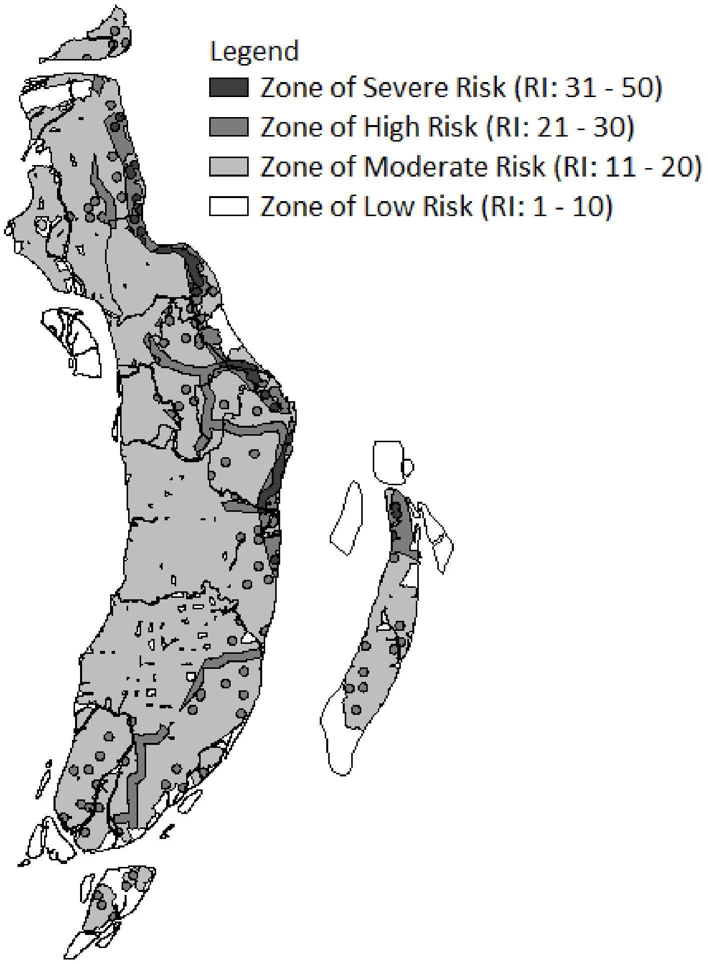

Fig 3.7: Risk Assessment Map of Bhola for Storm of 50 years Return Period

From the above figure (Fig 3.7) and table it can be concluded that for a storm of 50 years return period, the zones in Bhola district are mostly under moderate risk, though there are zones of high and severe risks in significant portion. Overall risk scenario has been shown in fig. 3.8.

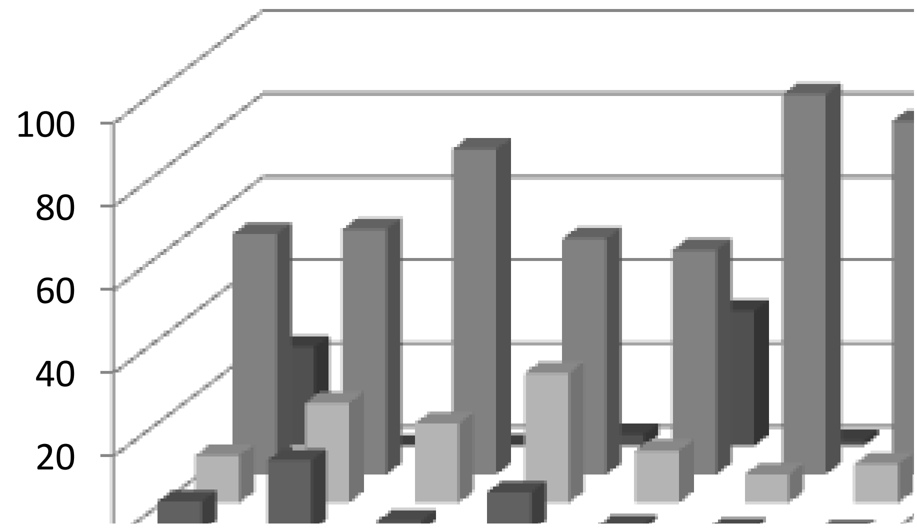

Fig. 3.8: Overall Risk Assessment Result

4. CONCLUSIONS

From the detailed discussions, the following conclusions can be made:

1. Due to the unique combination of a funneling coastal configuration, high population, density in coastal area, high tide and low flat coastal terrain in Bangladesh, it is highly susceptible to cyclonic storm surge.
2. For a storm surge of 1 in 50 year return period, Tazumuddin and Manpura will be inundated most.
3. Daulatkhan and Tazumuddin upazilas are at greater risk as their risk index values indicates.

4.1 Recommendations for Further Study

Some shortcomings could not be avoided while making every effort to achieve a successful finishing of the study. Some recommended measures thus should be taken to make this study a better one. The recommendations are –

- Use of detailed and updated land use data.
- Use of an advanced and updated GIS tool.
- Updating the hazard and vulnerability database and maps regularly so that risk assessment can be done to a greater accuracy.
- Assess the community preparedness of the coastal area for developing a proper warning system and evacuation routes.
- Assess the risk considering the public preparedness.

REFERENCES

Abdullah, M.M. and Hoque, M. M. 1995. "Prediction of Risk and Foundation of Counter Measures against Storm Surges for Chittagong Metropolitan Area." IFCDR, BUET, Dhaka.

Bangladesh Population Census, 2001. Bangladesh Bureau of Statistics.

Banglapedia, 2006.

Cultural Survey Report of Bhola, 2007.

Intergovernmental Oceanographic Commission. 2009. "Tsunami Risk Assessment and Mitigation for Indian Ocean".

Ouderm, P. 2007. "NOAA Researcher's Warning Helps Save Lives in Bangladesh", National Oceanic Atmosphere and Administration.

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ANALYTICAL ASSESSMENT OF GROUNDWATER RECHARGE POTENTIAL FOR THE GANGES-KOBADAK (G-K) IRRIGATION PROJECT AREA

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ABSTRACT

Quantification of groundwater recharge is a basic prerequisite for efficient groundwater management but its estimation is a complicated task in all ground water resources studies. This study is concerned with the development of a technique to estimate recharge of G-K (Ganges and Kabodak) irrigation project by the analysis of observed precipitation and water table fluctuations. In this study, for estimating ground water recharge, the well-known water table fluctuation technique has been implemented so that ground water recharge can be estimated using the shortest data available in hand. The estimated recharge database can be useful for numerical groundwater model study to estimate future groundwater use potential in the project area and also the runoff and the evapotranspiration obtained from the study can be used as an initial value for any detailed study in future. The overall objective of this study is to form a recharge database in the Ganges-Kobadak (G-K) Irrigation Project area in Bangladesh for estimating GW use potential in future.

Keywords: *G-K (Ganges and Kobadak), Aquifer, Groundwater recharge, Precipitation, Groundwater level, Water table fluctuation.*

1. INTRODUCTION

Groundwater (GW) is the important source of freshwater supply all over the world. Assessment of GW recharge and its use potentials is essential for the successful management of water resources systems. Comprehensive statistics on GW abstraction and use are not available, but it is estimated that more than 1.5 billion people worldwide rely on GW for potable water (Clarke et al., 1996). Since the world population is increasing continuously, more burdens on GW resources will be occurred, particularly in arid and semiarid areas. Long-term availability of GW supplies for growing populations can be ensured only if effective management schemes are developed and put into practice. Quantification of natural rates of GW recharge is imperative for efficient GW management. However, GW recharge processes are still one of the least understood, largely because recharge rates are difficult to directly measure as quantity of precipitation depends on several factors. Because of such factors, determining GW recharge and its use potentials is a great challenge in all GW studies.

GW recharge is also critically integrated with GW contamination. The important issues in areas of GW contamination as well as GW supply are the rate, timing and location of recharge. Generally, the probability for contaminant movement to the water table increases as the rate of recharge increases. Areas of high recharge are often equated with areas of high aquifer vulnerability to contamination (ASTM, 2008; US National Research Council, 1993). Locations for subsurface waste-disposal facilities often are selected on the basis of relative rates of recharge, with ideal locations being those with low aquifer vulnerability so as to minimize the amount of moving water coming into contact with waste.

From the mentioned above, it is clear that modelling of GW-flow are perhaps the most significant and useful steps for GW-resource management. Models are applied in both GW recharge management and its use potentiality.

2. METHODOLOGY

Many different methods and approaches exist for estimating GW use potentials. A complete understanding of the whole methods and the studies dictates that recharge estimation techniques be matched to conceptual models of recharge processes at individual sites to ensure that assumptions underlying the techniques are consistent with conceptual models.

2.1 System conceptualization

The development of a conceptual model of recharge processes is an important step in any recharge study. The conceptual model should be developed at the beginning of a study. It can be revised and adjusted as additional data and analyses provide new insights to the hydrologic system. Records of water level fluctuations in wells are worth the cost and trouble of collecting only if they are used as a basis for hydrologic interpretations. Although water level records have been vital to the reaching of conclusions regarding the occurrence and development of GW in specific areas, many such records still await interpretation. Similarly, a wealth of climatologic and other hydrologic data is in need of analysis (Korkmaz, 1988). In the wet period when precipitation occurs, the first rain wets vegetation and other surfaces and then begins to infiltrate into the ground. The first infiltration replaces soil moisture, and thereafter, the excess percolates slowly across the intermediate zone of saturation moves downward towards aquifer storage and consequently the water table goes up. In the dry period when evapotranspiration rate and GW withdrawal exceed the available moisture from precipitation, recharge to the water table is negligible and thus, GW levels decline (figure 1).

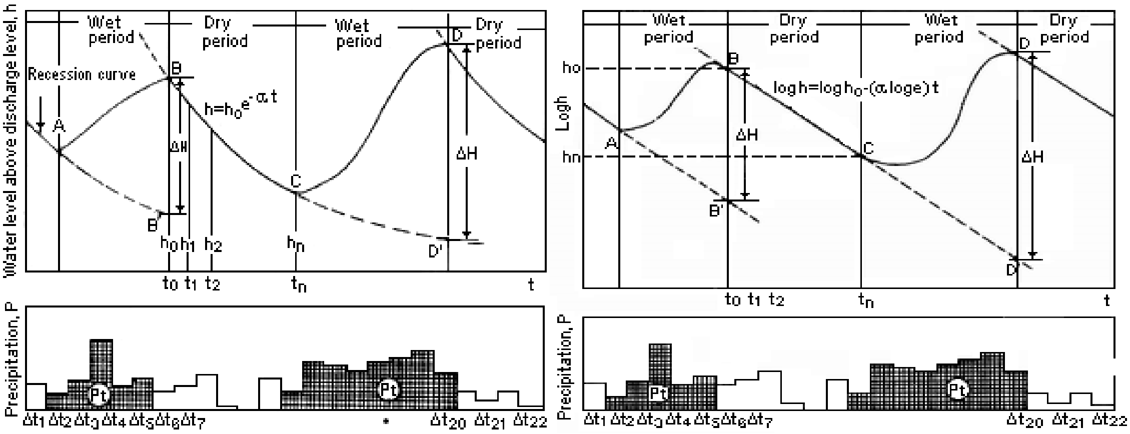

Figure 1: fluctuations in water level caused by recharge by precipitation (Adapted from Korkmaz, 1988)

In the general case it is possible to calculate the total level oscillation amplitude due to infiltration if the overall recession regime, i.e. the behavior of the aquifer without external recharge, is known. It is generally found that the component forming a GW hydrograph, including those from a GW system, frequently each had a recession that could be approximated by simple exponential relationships of the from (Figure 1):

$$h = h_o + e^{-at} \tag{1}$$

where h_o and h are the water level above discharge level at the beginning of the measurement period and after a certain time (*t*) respectively and a is known as coefficient of recession or discharge coefficient.

Like any other exponential formula, on a semi-logarithmic paper when water level above discharge level is plotted to the log scale and time to the arithmetic scale, the recession curves plot as nearly as straight lines (Figure 1). In the log system with base 10, the formula is converted into the following form:

$$\log h = \log h_o - 0.4343at \tag{2}$$

The shape of the recession curve depends on the water yielding properties of the aquifer material, the transmissibility and the geometry (Johansson, 1987).

The recovery of the water level, ΔH, under natural hydrological conditions is a mirror image of the recession curve. The recovery of the water level varies from year to year, depending on the amount of total precipitation (R_t) in wet period (Figures 1). GW levels are influenced by seasonal cycles in such factors as recharge from precipitation, evapotranspiration, and discharge from wells and show a seasonal pattern of fluctuations (Healy, 2010). The degree of correlation between fluctuations of GW level and fluctuation of total precipitation (R_t) in wet period furnishes a clue as to the freeness of the connection between recharge and total precipitation (R_t) in wet period. This study considers the direct estimation of recharge using recovery of the GW level (*ΔH*) and total precipitation (R_t) during wet period (Figure 2).

The line regression equation is

$$\Delta H = a + bR_t \tag{3}$$

Where ΔH is recovery of GW level, and R_t is total precipitation during the wet period, a & b are the regression coefficients.

Precipitation intercept, R_e is intersection of the total precipitation recovery straight line with zero-total precipitation axis (Figure 2) and it represents the amount of surface runoff and evapotranspiration for the same period. Recovery or recharge from precipitation is a function of the amount of total precipitation (R_t). If the intercept is Re, the recharge (R_s) is estimated as

$$R_s = R_{tc} + R_e \tag{4}$$

Where, R_{tc} is the result of the computation of the total precipitation by means of equation (3) during the wet period in a year.

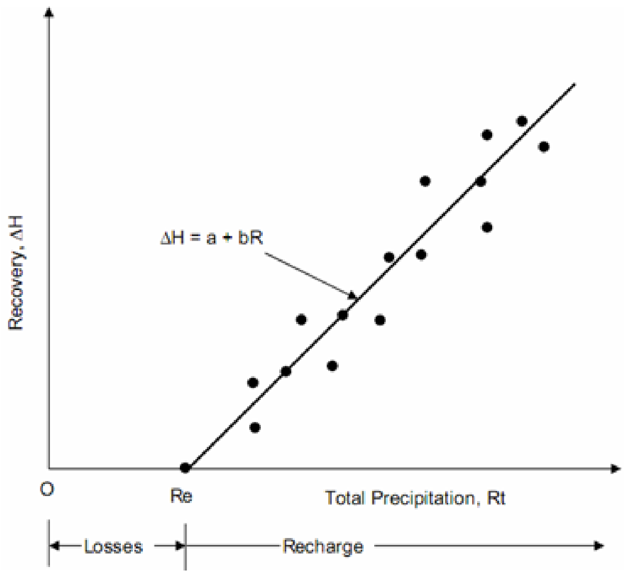

Figure 2: Relationship of total precipitation in wet period and recovery of the aquifer water level.

2.2 Estimation of GW Recharge

In this paper, a modified GW fluctuation technique is represented for estimating GW recharge in shallow depth based on a statistical relation between the precipitation and water levels. The analysis is based on the original conditions, which are unaffected by heavy pumping. Observed monthly precipitation and weekly GW levels are the only data required to carry out the study.

By analyzing the monthly precipitation and water level records belonging to the period, the amount of rainfall causing a rise in water level is determined. These rainfall amounts are called "total precipitation (R_t)" causing the rise in water level. The water level rise caused by this total precipitation is termed as "water level rise due to GW recharge (ΔH) and can be found from the graph directly. This procedure is repeated year after year for the period of record from 1999 to 2006 for all the individual well station and the relation between these two variables (R_t) and ΔH is established by using simple statistical technique. ΔH is the function of the precipitation amount above a threshold rainfall value, which represents the precipitation causing surface runoff, evapotranspiration, and subsurface flow or only one of them depending on the local hydrological conditions in the study area. It is obtained by setting ΔH = 0 in the regression equation, which is represented in figure 1. The recharge amount for each year is then calculated by subtracting the threshold precipitation from the total precipitation causing water level rise in that year.

3. RESULTS AND DISCUSSION

The Ganges-Kabodak (GK) irrigation project supplying water in five districts of the country's south-western belt for boro cultivation has been as case study here. The districts include Kushtia, Jhenidah, Chuadanga, and Magura. The project linking the rivers Gorai and Padma will supply water to produce 275,000 tonnes of paddy on 116,000 hectares of land in the region. The project is using 15 pumps to lift water with a capacity of 153 cusec water from the two rivers through an intake channel to the project areas. According to official sources, the project authorities had been supplying water only for two seasons--kharif-1 (March-June) and Kharif-11 (July-November) since the beginning of the project in 1961. A new crop season has been introduced, namely Kharif-111 (December-March), in the irrigation system last year. During the dry season, the water level of river Padma go much below and up to 186000 cubic meters of silt has to be dredged annually from 850 meter stretch of canal near the Hardinge Bridge.

As this project covers about 37 well points under several well stations to accomplish its goal of supplying water for irrigation purpose, here in this paper, due to analytical assessment under limited paper size, one of the well points methodology of determining GW recharge potentialities has been demonstrated and others estimated recharge potentials has been shown and the selected well point is **GT5557001** which is under the district of Magura.

The GW levels naturally fluctuate in response to a sequence of climatic events and to constraints imposed by the hydro-geologic and topographic characteristics (figure 3). The figure demonstrates that the aquifer is recharged in the wet period due to the excess rainfall and discharged in the dry period as there is not much rainfall available in that time. In addition, evapotranspiration and water withdrawal takes place, which cause the depletion of the water table. GW recharge is largest in the wet period, especially in the monsoon, when plants are dormant and evapotranspiration rates are comparatively less. In the dry period, when evapotranspiration rates exceed the available moisture from precipitation, recharge to the water table is negligible and thus, GW levels decline (figure 3).

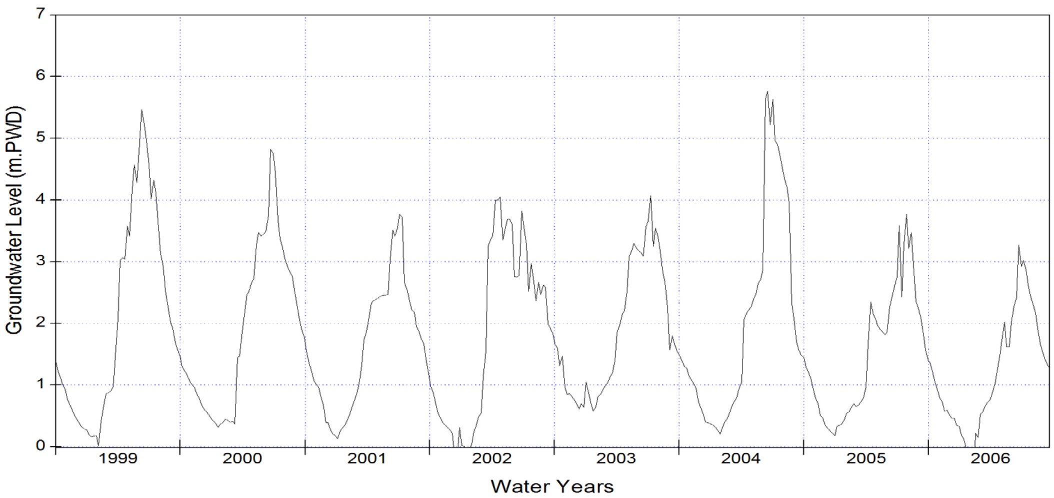

Figure 3: Weekly GWL hydrograph for 1999 to 2006 in the GT5557001

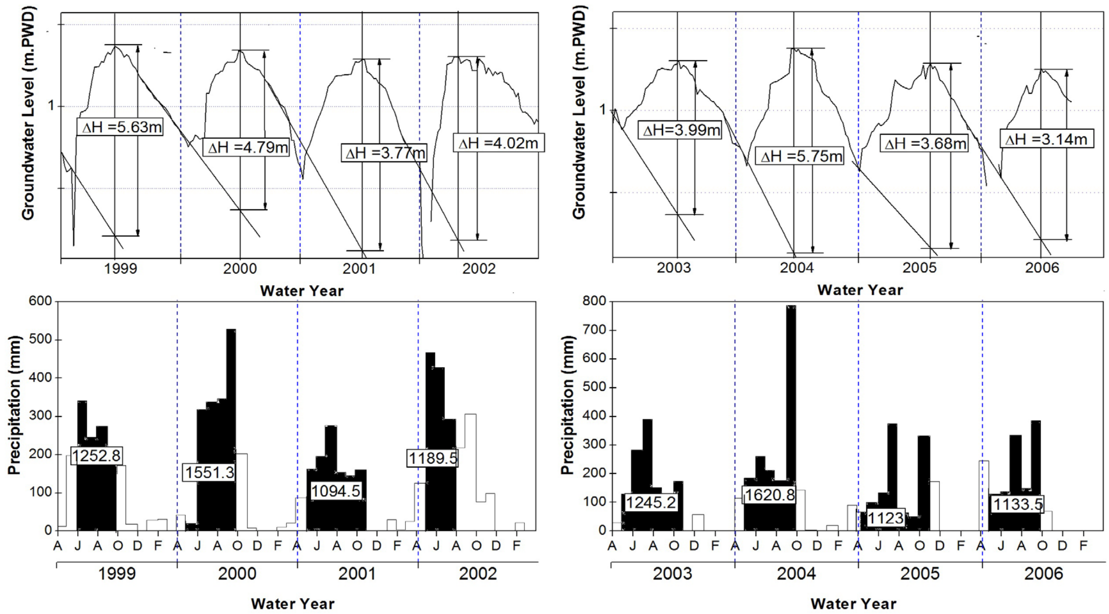

Figure 4: Fluctuations in water levels caused by recharge from precipitation for GT5557001 monitoring well.

The recovery of the water level (ΔH) and the total precipitation (R_t) during the wet period in a year for water years from 1999 to 2006 are determined (Figure 4) and a linear regression equation during the period is established (Figure 5).

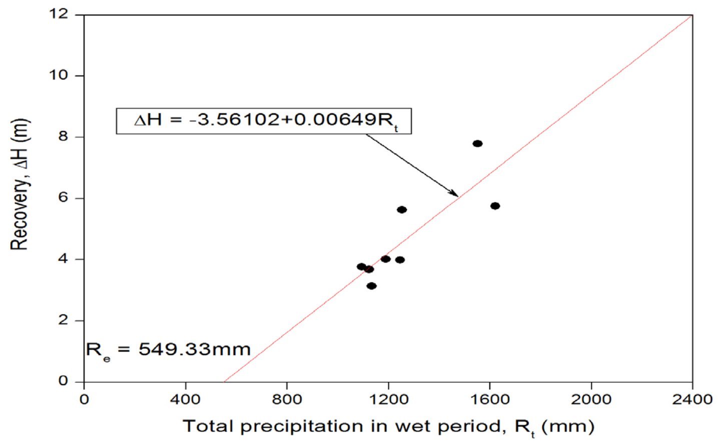

Figure 5: Relation of total precipitation in wet period and GW recovery.

The results of linear regression analysis are presented in table 1.

Table 1: Results of linear regression analysis

Type of relation	**No. of observations**	**Correlation coefficient, r**	**Standard error of estimate**	**Regression equation**
Total precipitation GW level recovery	8	0.83777	2.23733	ΔH = -3.56102+0.00649 R_t

Table 2: Summary of recharge computation

Water Year	Annual precipitation, R_y (mm)	Recovery of GW level, ΔH (m)	Computed total precipitation, R_{tc} (mm)	Recharge, R_t (mm)
1999	1480.80	5.63	1416.181818	866.85182
2000	1863.80	7.79	1349.001541	799.67154
2001	1212.50	3.77	1129.587057	580.25706
2002	2071.00	4.02	1168.107858	618.77786
2003	1353.20	3.99	1163.485362	614.15536
2004	1882.50	5.75	1434.671803	885.3418
2005	1405.00	3.68	1115.719569	566.38957
2006	1448.40	3.14	1032.514638	483.18464

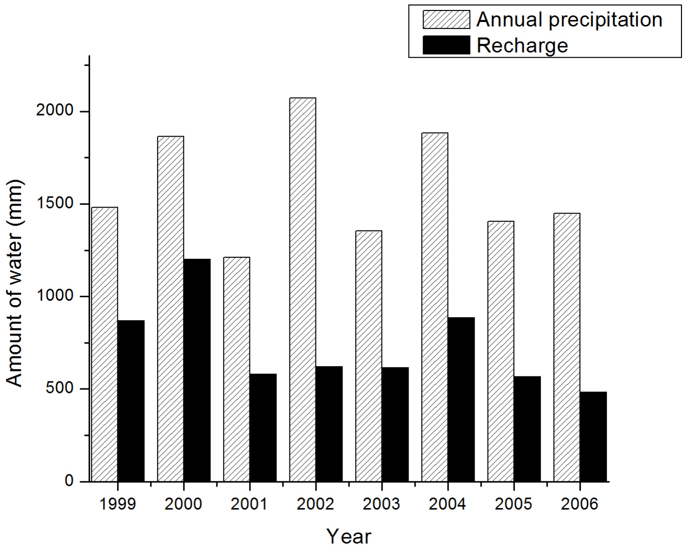

Figure 6: Estimated GW recharge for 8 years in the well point GT5557001

The regression equation is ΔH = -3.56102+0.00649 R_t . The average precipitation intercept (Runoff) of the aquifer is found as 549.33mm (Figure 5). The computed values (R_{tc}) of total precipitation by means of the regression equation for water years from 1999 to 2006 are summarized in table 2. The results of the computation of the recharge (R_t) from precipitation of this aquifer by means of equation $R_s = R_{tc} + R_e$ for the whole 8 years period (1999-2006) ranges from 483.1846379mm in 2006 to 885.3418028mm in 2000 (Figure 6). The annual average recharge during the period from 1999 to 2006 is 676.8287057mm. This is about 42.58% of the average annual precipitation (1589.65mm) in this well point.

Similarly the GW recharge of various well points under several well station of corresponding region have been determined and due to ease of the assessment and limitation of paper size, the other well point's GW recharge estimation procedure, has been taken as ellipsis, as it's analogous to the previous procedure. But the final outcomes of several well points are demonstrated as followed. For the ease of representation of the results the well points and well stations are subdivided under several district and region.

The regions that the G-k irrigation project covers are Alamdanga, Amla, Kushtia, Magura and Shailkupa. The corresponding outcomes of several well points under these zones are demonstrated respectively.

3.1 Alamdanga

Alamdanga is an upazilla of Chuadanga district in the division of Khulna, Bangladesh. It covers an area of 360.4 square kilometres (139.2 sq mi).Alamdanga is located at 23.7583°N 88.9500°E . It has 44,699 households and total area 360.4 km².

Alamdanga upazilla contribute a station for G-K irrigation project and the station id is R452 Alamdanga. There are about five effective well points in Alamdanga upazilla. And those are GT1815001, GT1815004, GT1815501, GT4414001 and GT4414002. The estimated GW recharges of these well points are demonstrated below in table 3.

Table 3: Estimated GW recharge of several well points of Alamdanga

Water Year	Recharge of Well points of Alamdanga (mm)				
	GT1815001	GT1815004	GT1815501	GT4414001	GT4414002
1999	1232.79	1979.50	1391.31	1360.57	2168.18
2000	1353.36	1545.24	941.49	1079.58	1458.30
2001	945.71	1381.89	1598.93	666.36	715.41
2002	1428.96	1692.65	1481.28	1327.51	1375.76
2003	869.15	1051.22	1512.42	1509.33	1293.21
2004	1792.60	1883.88	657.75	1773.79	1904.04
2005	1926.57	2003.41	2007.23	2079.58	2069.13
2006	2199.30	1891.85	2221.76	1881.23	1904.04

3.2 Amla

Amla is an upazilla of Chuadanga district in the division of Khulna, Bangladesh. Amla upazilla contribute a station for G-K irrigation project and the station id is R214 Amla. There are about eight effective well points in Kushtia district. And those are GT5015007, GT5015009, GT5094027, GT5094028, GT5094030, 5094031, 5094511 and 5094512. The estimated GW recharges of these well points are demonstrated below in table 4.

Table 4: Estimated GW recharge of several well points of Amla

Water Year	Recharge of well-points of Amla (mm)							
	GT5015007	GT5015009	GT5094027	GT5094028	GT5094030	GT5094031	GT5094511	GT5094512
1999	1552.62	1573.60	1746.23	1754.73	1267.17	1579.78	1702.01	1536.84
2000	1109.37	1094.73	1107.18	1062.79	1047.99	1111.03	1112.82	1066.64
2001	1309.37	1352.94	979.37	1238.15	1274.02	1400.09	1393.90	1185.84
2002	1466.13	1310.69	752.16	825.82	1137.03	1361.03	788.50	1142.80
2003	1022.87	1000.83	610.15	797.39	767.17	1048.53	1058.77	752.07
2004	1585.05	1658.11	1689.42	1659.95	1465.80	1587.59	1712.82	1662.67
2005	547.21	1827.12	1973.44	1830.56	1883.61	1782.91	2015.52	1834.85
2006	682.35	681.58	1078.78	982.22	1034.29	907.915	1102.01	1073.26

3.3 kushtia

Kushtia district has an area of 1621 square kilometers and is bounded by Rajshahi, Natore and Pabna districts to the North, by Chuadanga and Jhenaidah districts to the South, by Rajbari district to the east and by West Bengal and Meherpur district to the west. Kushtia district contribute a station for G-K irrigation project and the station id is R19 Kushtia. There are about five effective well points in Kushtia district. And those are GT5079024, GT5079026, GT5079505, GT5071506 and GT5071507. The estimated GW recharges of these well points are demonstrated below in table 5.

Table 5: Estimated GW recharge of several well points of Kushtia

Water Year	Recharge of Well points of Kushtia (mm)				
	GT5071506	GT5071507	GT5079024	GT5079026	GT5079505
1999	1804.03	1964.49	1501.86	1885.81	1913.10
2000	1698.77	1528.49	1359.97	1469.97	1324.49
2001	1407.27	1673.82	1684.30	1702.64	1577.66
2002	1609.70	698.03	1164.03	1370.96	1299.18
2003	136.01	1009.45	1224.84	999.67	1223.23
2004	1548.97	1736.11	1839.70	1836.31	1818.16
2005	1293.91	1092.50	934.30	1628.37	805.51
2006	1261.52	864.12	927.54	1167.99	1261.20

3.4 Magura

Magura is a district of Khulna Division.in the South-Western Bangladesh. Its a**rea is** 1048 km^2.**It** is bounded by Rajbari district on the north, Jessore and Narail districts on the south, Faridpur district on the east and Jhenaidaha district on the west. Magura district contribute a station for G-K irrigation project and the station id is R460 Magura. There are about seven effective well points in Magura district. And those are GT5557001, GT5557002, GT5557004, GT5595009, GT5595010, GT5595011 and GT5595012. The estimated GW recharges of these well points are demonstrated below in table 6.

Table 6: Estimated GW recharge of several well points of Magura

Water year	Recharge of Well-points of Magura (mm)						
	GT5557001	GT5557002	GT5557004	GT5595009	GT5595010	GT5595011	GT5595012
1999	866.85	1459.56	1415.48	1420.74	1402.04	699.53	1301.29
2000	799.67	1601.79	1636.41	1640.879	1683.78	1704.75	1838.08
2001	580.26	1184.01	898.04	1131.43	1072.67	489.14	960.64
2002	618.78	1384.01	1229.43	1200.62	1564.73	2032.03	1838.08
2003	614.16	1215.12	1188.74	1081.12	1191.72	1143.70	1249.68
2004	885.34	1557.34	1851.53	1867.28	1540.92	1634.62	1290.97
2005	566.39	1188.45	816.65	1043.38	1167.91	816.42	650.95
2006	483.18	890.68	1025.95	1074.83	1223.46	1050.19	888.38

3.5 Shailkupa

Shailkupa Upazilla contribute a station for G-K irrigation project and the station id is R463 Shailkupa. There are about nine effective well points in Magura district. And those are GT4480014, GT4480015, GT4480016, GT4480017, 4480018, GT4480019, GT4480020, GT4480021 and GT4480022. The estimated GW recharges of these well points are demonstrated below in table 7.

Table 7: Estimated GW recharge of several well points of Shailkupa

Water Year	Recharge of Well-points of Shailkupa (mm)								
	GT4480014	GT4480015	GT4480016	GT4480017	GT4480018	GT4480019	GT4480020	GT4480021	GT4480022
1999	1549.10	1792.86	1752.77	1656.54	1740.33	1742.28	1808.6	1813.8	1855.1
2000	1603.52	1987.33	1863.88	1778.58	1857	1801.11	1345.9	1748.4	1841.4
2001	1283.798	1339.08	1271.29	1470.10	1240.33	634.44	1286.2	841.8	1355.1
2002	1793.99	1938.72	1873.14	1998.92	1957	2006.99	1987.7	2225.0	1795.5
2003	1413.04	1241.84	1345.36	1412.47	1557	1477.58	1629.5	1589.5	1488.2
2004	773.59	905.13	882.40	914.17	907	899.15	905.6	580.1	864.3
2005	1072.91	1193.23	1400.92	1344.68	1398.67	1173.66	1398.2	1206.3	1300.1
2006	970.86	1031.16	1123.14	1093.83	1123.67	683.46	1092.2	1122.2	1079.9

4. CONCLUSIONS

The goal of this study has been a synthesis of the results which can be obtained by analysis of water level fluctuations. The present work suggests the recharge estimation of G-K (Ganges and Kobadak) irrigation project. The irrigation project area consists of several well stations (which have been described earlier) from which the irrigation areas receive water. And the recharge of the GW in this area is from precipitation. It is clear from the previous discussion that the GW recharge of the areas takes place in the wet period, especially in the monsoon (wet period) season due to heavy rainfall when plants are dormant and the evapotranspiration rates are comparatively less.

In this study the GW fluctuation method has been used because of the ease of this method with shortest possible data and information that are available in hand. From the analysis of all the well points and station of the project area through water level fluctuation method it is undoubtedly visible that the project areas are recharged from precipitation and the amount of this precipitation have been shown in the results and discussion chapter. Nevertheless, by using this method other hydrologic variables such as evapotranspiration and runoff can be obtained indirectly, which can be used as an initial value for any detailed study in future. Eventually from the whole system analysis, amount of GW recharge in every single station and areas have been estimated. The estimated recharge database can be useful for numerical GW model study to estimate future GW use potential of G-K (Ganges and Kobadak) Irrigation Project.

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REFERENCES

ASTM (2008). Standard guide for selection of methods for assessing ground water aquifer sensitivity and vulnerability. ASTM D6030–96(2008). American Society of Testing Materials.

Clarke, R., Lawrence, A. and Foster, S. (1996). Groundwater: A threatened resource. Nairobi, Kenya: United Nations Environment Programme Environment Library No. 15.

Healy, R.W. (2010). Estimating Groundwater Recharge, Cambridge University Press, New York, USA.

Johansson, P.O. (1987). "Estimate Of Groundwater Recharge in Sandy Till with Two Different Methods using Groundwater Level Fluctuations", Journal of Hydrology, Vol. 90(3-4), 183-198.

Korkmaz, N. (1988). "Estimation of Groundwater recharge from Water Level and Precipitation Data." Journal of Islamic Academy of Sciences Vol. 1(2), 87-93.

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ESTABLISHMENT OF CO-RELATION BETWEEN REMOTE SENSING BASED TRMM DATA AND GROUND BASED PRECIPITATION DATA IN NORTH-EAST REGION OF BANGLADESH

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ABSTRACT

Estimation of precipitation data is crucial due to high dependency of water resources management, crop yield assessment, flood and drought monitoring on precipitation data. Devastating crop yields which limit human consumption due to drought are the serious consequences of extended absence of rainfall. On the other hand excessive rainfall causes flooding and severe damage to property and lives. Bangladesh is also heavily dependent on rain-fed agriculture, hence excess or insufficient rainfall can be devastating to the national economy and the livelihood of its people. Flash flood occurs in almost every year during pre-monsoon to post-monsoon months (March-May) in North East region of Bangladesh. But ground based rainfall data available for the North East region is not so consistent and reliable always due to excessive missing values and errors in the collected data. This could be a limitation in the simulation of flood water levels for this region. Therefore, possibility of improving the understanding of rainfall distribution by incorporating the rainfall estimates from a spatially distributed, comprehensive and systematic source like the Tropical Rainfall Measuring Mission (TRMM) is explored in this paper. TRMM is specially designed to increase the extent and accuracy of tropical rainfall measurement. In this paper, eight years of data from 1999 to 2006 of TRMM 3B42 version 6 (V6), which allows aggregating the values to a daily, monthly and yearly resolution and the Bangladesh Water Development Board (BWDB) rain gauge network are analysed to understand the rainfall characteristics over North-East region of Bangladesh. Daily rainfall measured by TRMM 3B42 version 6 is compared to that of rain gauge values from pre-monsoon to post-monsoon months (March–May), which results following statistical measures: BIAS, MAD and RMSD. Finally, a co-relation is established between remote sensing based TRMM data and ground based precipitation data for the North-East region of Bangladesh by analysing those statistical parameters. Results of this study will be helpful to update and complete the existing rainfall database, simulate the flash flood in time and enrich more rainfall relevant research studies in future for the North-East region of Bangladesh.

Keywords: *TRMM, Rain gauge data, Flash flood, Inconsistent rainfall data, North-East region*

1. INTRODUCTION:

Precipitation measurement is extremely vital for water resource management because of the dependency on the meteorological characteristics in case of crop yield assessment, flood and drought monitoring. Popularity of using remote sensing measurements data have now been immensely increased in case of understanding of spatio-temporal variation of meteorological parameters specially the areas where data availability is scarce. To meet that demand, an initiative of the US National Aeronautics and Space Administration (NASA) and the Japanese Aerospace Exploration Agency (JAXA), the Tropical Rainfall Measuring Mission (TRMM) (Simpson et al., 1988; Kummerow et al., 1998), is introduced to the use of satellite-based rainfall products in hydrological studies (http://trmm.gsfc.nasa.gov/). As by the name, the mission actually covers only the tropical zone, i.e., between the latitudes 50^0N and 50^0 S. Its current spatial resolution is 0.25^0 (Arias-Hidalgoet al, 2012).

The North East Region of Bangladesh has unique hydrological characteristics. Special climatic characteristics like twice-yearly reversal of monsoon wind movement and widely diverse topographical features set this region to be highly prone to floods. Within aerial coverage of around 175 km, annual rainfall varies from 2200 mm along the western boundary to 5800 mm in its north east corner and is as high as 12000 mm in the headwaters of some hilly catchments extending to India. Enormous amount of runoff is accumulated within a very short period of time, especially during the pre-monsoon season so that flash floods (pre-monsoon flood) are generated in the steep, upland catchments adjacent to the region in India. Large amount of water carried by the trans-boundary rivers deluge the harvesting of the main Boro crop in protected areas and inundation exceed design level of submergible embankments. Excess rainfall in the upstream hilly areas and subsequent runoff, sedimentation in the rivers and loss of navigability, construction of unplanned road and water management infrastructures, deforestation and hill cuts, land slide, improper drainage and last but not the least the effect of climate variability can be treated as the main reason of the devastation caused by flash floods. As the distribution of rain gauge measurement locations are not sufficient to capture the spatial variation in rainfall due to various topographical features, combination of TRMM techniques with rain gauge measurements can support the estimation of acceptable precipitation values in this region. This method is more applicable for shorter periods, such as daily, five day (pentad) or ten day (decade) rain rates.

In this study, the statistical measures BIAS, MAD, RMSD are calculated from the analysis which establish a co-relation between remote sensing based TRMM data and ground based precipitation data for the North-East region of Bangladesh by analysing those statistical parameters. This result will be able to form an evaluation of applicability of satellite rain estimates in Bangladesh on agricultural and hydrological applications which will be helpful to simulate the flash flood in time.

2. STUDY AREA AND DATA SOURCES

The study area for the present study has been chosen to cover the North East region of Bangladesh. Figure 1 illustrates the study area within Bangladesh. Figure 1 shows the boundary of the seven districts in the Haor region (Sylhet, Sunamganj, Moulvibazar, Habiganj, Netrokona, Brahmanbaria and Kishoregonj) of Bangladesh. The precipitation data for the study has been collected from two sources, TRMM and rain gauge network of Bangladesh Water Development Board (BWDB). Here, eight years of data from 1998 to 2006 of TRMM data product 3B42 version 6 (V6), which allows aggregating the values to a daily, monthly and yearly resolution and the daily rainfall data collected at rain gauge locations are analysed to assess the variation between two sources and their usability in representing the rainfall characteristics over North-East region of Bangladesh.

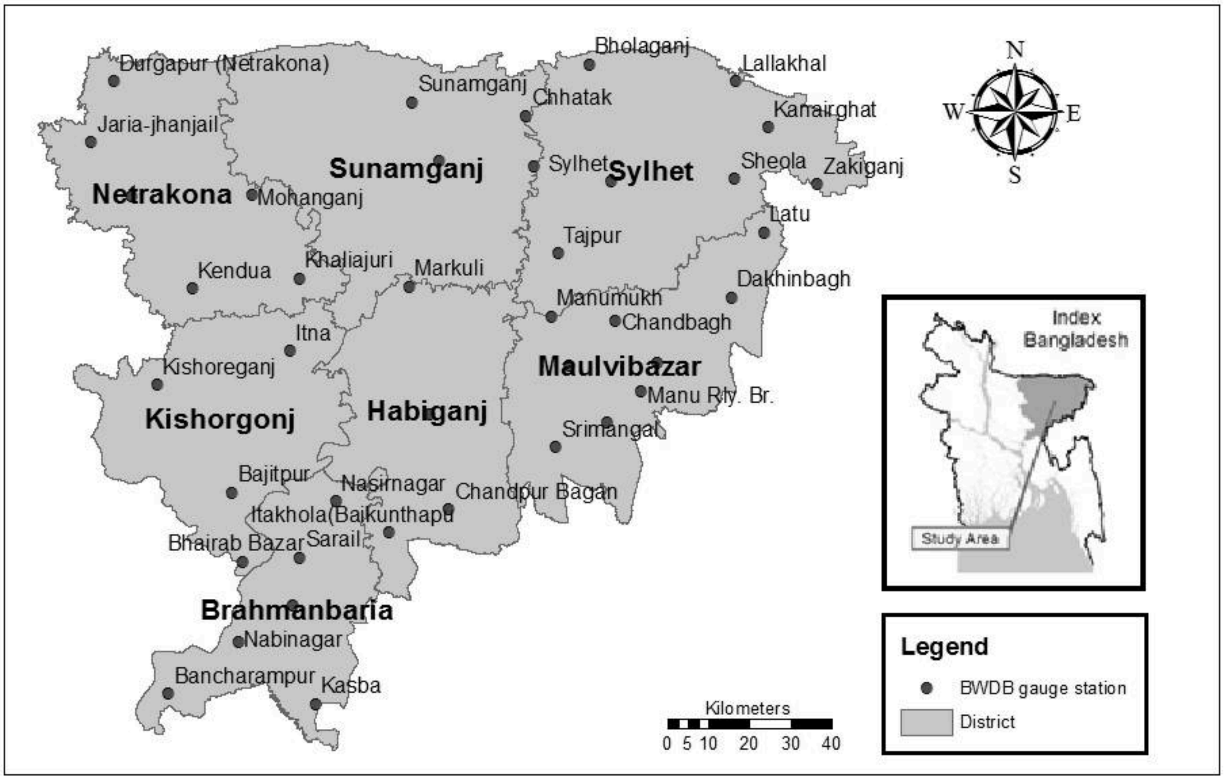


Figure 15: Map of study area

The following sections describe the data sources used in the study:

2.1 Rain Gauge dataset (BWDB):

The rain gauge data are obtained from the Bangladesh Water Development Board (BWDB). Eight years (1999-2006) of daily rain rates are collected from rain gauge stations over the Northeast region of Bangladesh. Figure 1 shows the locations of the gauge stations. The dataset is made for the North East Region.

2.2 TRMM 3B42 Data:

This database is compiled from the 3B42 product of the Tropical Rainfall Monitoring Mission (TRMM).The data represent three-hourly rainfall estimates for 0.25 degree cells for all land areas from 50 degrees North to 50 degrees South supplied in annual time series files for each grid cell from 1998-2006.Table 1 shows the data sets and their characteristics used in this study.

Table 1: Characteristics of the data sets used in this study

Data Set Name	Spatial resolution	Temporal Resolution	Data available duration
Rain Gauges	-	Daily	Jan 1957- Dec 2008
TRMM 3B42	0.25x0.25 degree	3-hourly	Jan 1998- Dec 2006

3. METHODOLOGY

The purpose of this study is to compare the rainfall estimation of TRMM rainfall products with Bangladesh Water Development Board (BWDB) (Ground-based data) over North West region in Bangladesh. Among all the available TRMM data products, the 3B42 was used in this study as recommended by previous researchers (Winsemius, 2009; Dinku et al., 2010; Almazroui, 2011; Vernimmen et al., 2012). The available three-hourly data throughout a time span of 8 years (1999–2006) were aggregated to monthly and yearly resolutions. Daily rainfall measured by TRMM 3B42 V6 is compared to that of rain gauge values from pre-monsoon to post-monsoon months (March–May).In order to select the influence zones of each BWDB rain gauge, the Thiessen polygon method have been used. Later the TRMM data point and BWDB station residing in the same Thiessen polygons were compared to establish a co-relation. Figure 2 shows the locations of the TRMM and BWDB station locations. Figure 2 also shows the selected stations for comparison and the Thiessen polygons around each of the stations. The overall methodology has been illustrated in Figure 3.

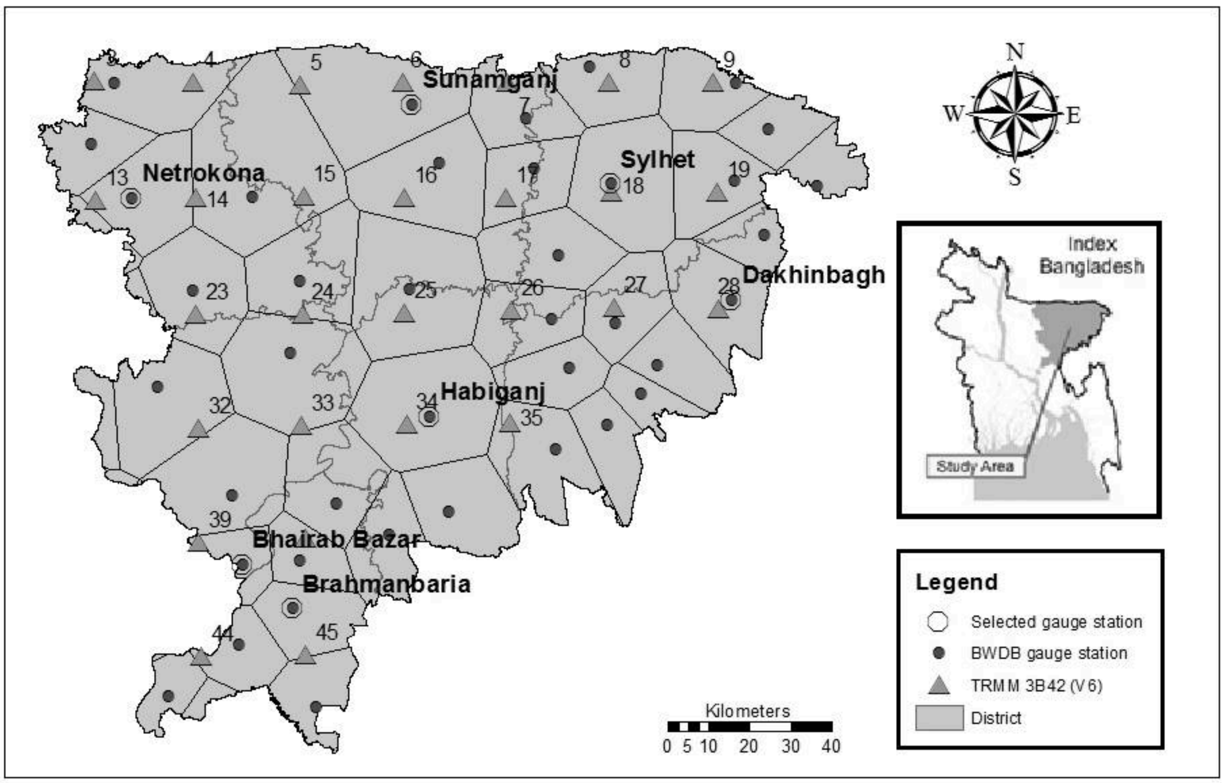


Figure 16: Location of TRMM, BWDB data points and selected rain gauges

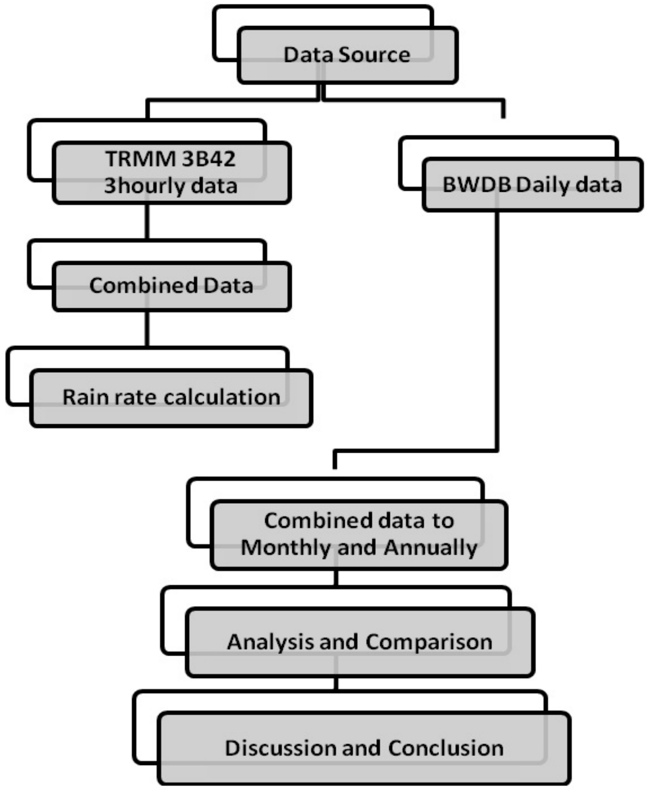


Figure 3: Flow chart of methodology

For the study ArcGIS 10.1have been used for data preparation and analysis that is shown in figure 2. Annual rainfalls from rain gauges and the 3B42 data over the time span were computed at their respective measurement points. To compare these rain estimates, the following statistical measures are used:

(1) Bias

$Bias = 1/n \sum_i (TRMM_i - BWDB_i)$

(2) Root mean square difference (RMSD)

$RMSD = 1/n \sum_i \sqrt{} (TRMM_i - BWDB_i)^2$

(3) Mean absolute difference (MAD)

$MAD = 1/n \sum_i \left| TRMM_i - BWDB_i \right|$

Where n is the total number of samples, i= 1, 2, 3… and TRMMi is the TRMM 3B42 v6 rain rate and BWDBi is the BWDB rain gauge data.

4. RESULTS

The results of the analysis is prepared in two sections, Monthly and annual analysis and comparison. These are described as follows:

4.1 Monthly Analysis

Figure 4 shows the monthly cycle of rainfall for the stations where horizontal axis denotes months and left vertical Y axis shows MAD and RMSD while right vertical axis shows BIAS values for TRMM 3B42 against BWDB rain gauge data.

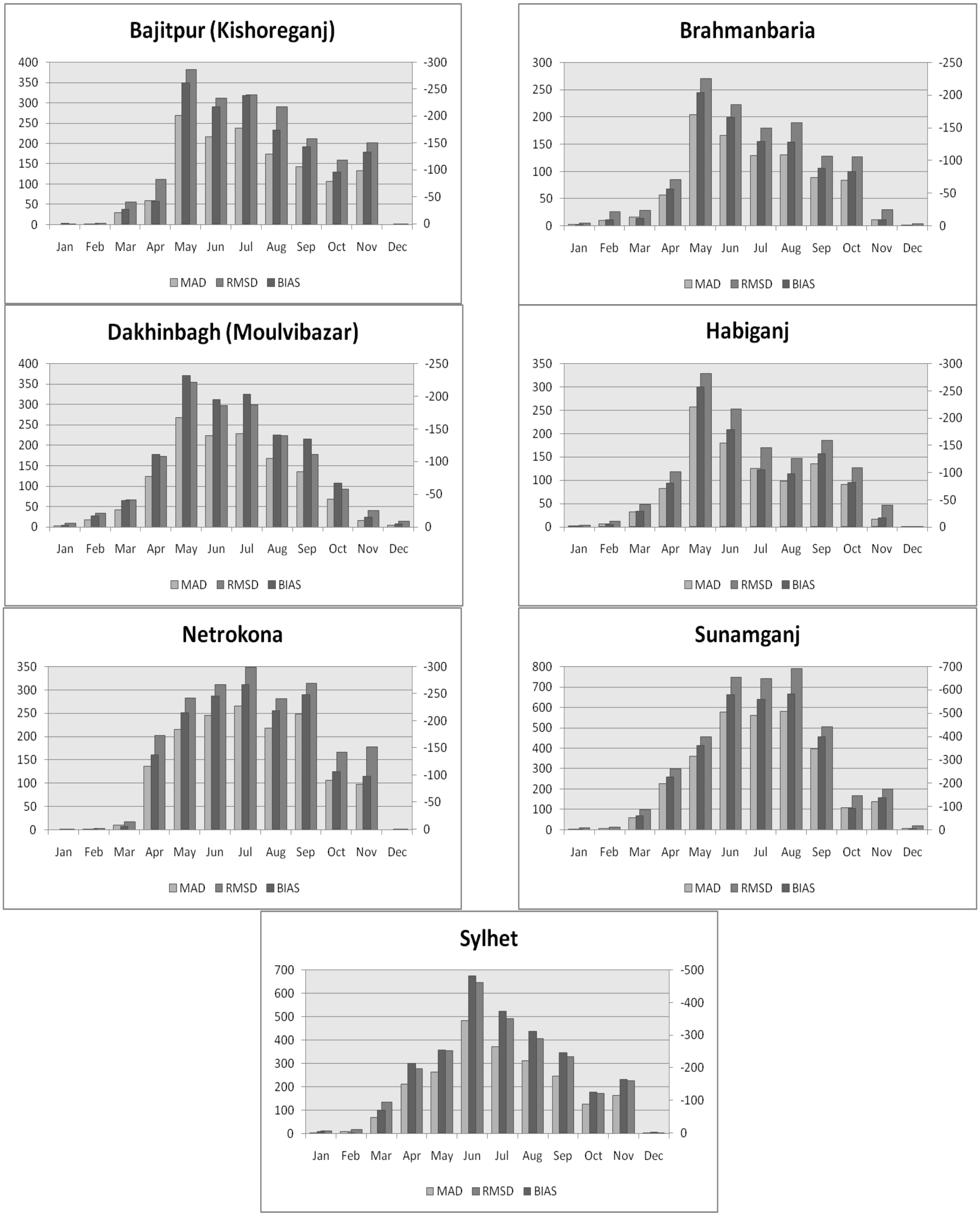


Figure 4: Monthly comparison of TRMM and BWDB

The data are computed from 8-year data (1999-2006). All estimates show peaks in May to August, respectively, during the rainy season. In Bajitpur station of Kishoreganj there is a comprehensive change in MAD from May to November and a slight change in March and April. During dry period there is hardly any change noticed. In May, the MAD value is the highest around 260 mm and minimum in December. The range of MAD value is found as 25-260 mm.

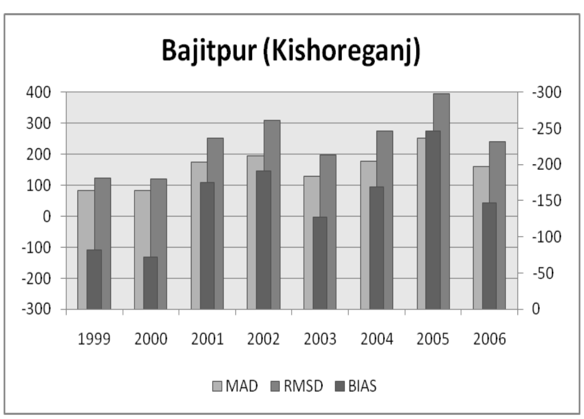
Similar changes have been seen during the RMSD calculation. There is a less significant change during dry season while from March to November it varies significantly. From May to September the values of RMSD fluctuates the most. The highest value has been seen in May around 380mm, while the lowest one is in December. RMSD values changes in a range of 55-380 mm.

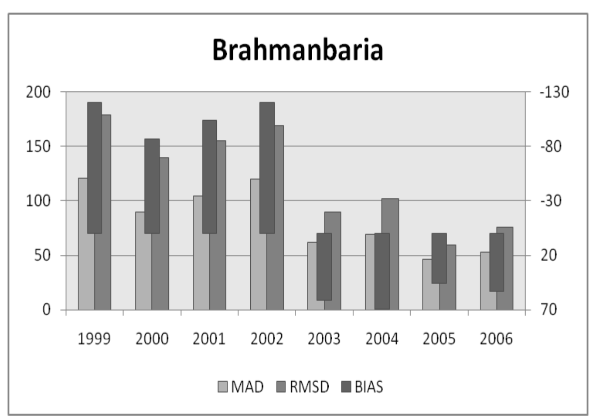
The bias analysis shows similar changes occurring during the rainy season and a little change has been seen on dry season. During rainy season the bias values are varying the most. The highest values of bias are seen in May around -250mm and the lowest value has been seen during dry season. Similar situation has been seen on the other rainfall stations.

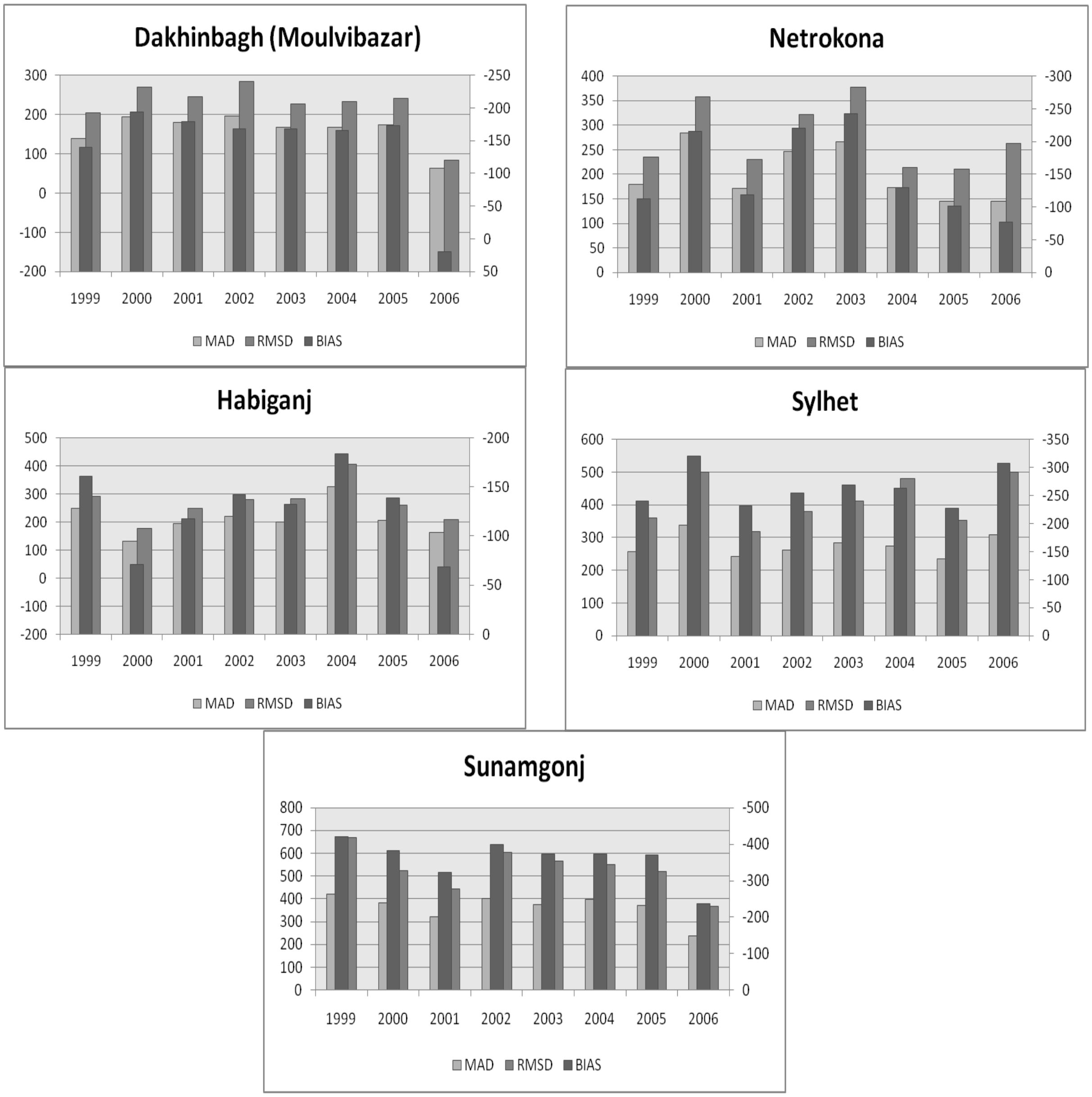
4.2 Annual Analysis

This section focus on yearly comparison of TRMM 3B42 and BWDB data compared over North East region of Bangladesh. To understand the rainfall over North East region, this section determine rainfall average over Bangladesh in mm/year units that is shown in figure 5where horizontal axis denotes years and left vertical Y axis shows MAD and RMSD and right vertical axis shows BIAS values. TRMM and BWDB rain gauge data are computed from 8-year data (1999-2006). Yearly rainfall results are quite dissimilar than monthly results. At Bajitpur station in Kishoreganj the highest MAD value is seen in year 2005 which means at that year BWDB rainfall value was a bit higher than TRMM value. The highest MAD value is seen in 2005 while the lowest value is seen in year 1999. Similarly highest RMSD value is found in year 2005 and the lowest value is found in 1999. In BIAS analysis results fluctuate than MAD and RMSD. The highest value of BIAS is found in 2005 and the lowest is found in 2000.The range of values of MAD, RMSD and BIAS are around 80-250 mm, 110-400 mm and -72 to -246 mm. Dakhinbagh and Habiganj stations results are quite similar with Bajitpur station.

On the other hand the results of Brahmanbaria station are slightly different than the other stations mentioned above. The highest MAD and RMSD values are found in 1999 while the lowest values are found in 2005. But the BIAS value seems quite a bit fluctuating after 2003.The ranges of MAD, RMSD and BIAS are around 40-130 mm, 60-180mm and 60-180 mm. At Netrokona, station the results fluctuates most and at Sylhet and Sunamgonj deviation of the results are quite linear.







5. Figure 5: Annual comparison of TRMM and BWDB

6. CONCLUSIONS

The comparisons of 3B42 3-hourly products and BWDB gauge data have shown higher bias, RMSD and MAD.

The analysis rainfall over North East region of Bangladesh show higher values in monsoon and pre-monsoon season while dry season shows low. The rainfall estimated by remote sensing show lower rain rate than ground-based data. The possible causes are: the nature of temporal resolution of TRMM satellites especially in monsoon season, TRMM strip lines locate at the equator, the higher latitude than 5 degrees are measured in the form of diagonal line around triangular areas but this technique still has some errors when applied in subtropical region, the spatial resolution at 0.25 degree but convective clouds are usually smaller than that measurement point. For further study, investigation can be done with the 3 hourly and daily TRMM data to learn the rainfall pattern by using different statistics.

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REFERENCES

Adler, R. F., Huffman, G. J., Bolvin, D. T., Curtis, S., and Nelkin, E. J.: Tropical Rainfall Distributions Determined Using TRMM Combined with Other Satellite and Rain Gauge Information, J. Appl. Meteorol., 39, 2007–2023, 2000.

Almazroui, M.: Calibration of TRMM rainfall climatology over Saudi Arabia during 1998–2009, Atmos. Res., 99, 400–414, 2011.

Arias-Hidalgo, M., Bhattacharya, B., Mynett, A.E., and Griensven, A. van : Experiences in using the TRMM data tocomplement rain gauge data in theEcuadorian coastal foothills, Discussion paper, Hydrol. Earth Syst. Sci. Discuss., 9, 12435–12461, 2012.

Arias-Hidalgo, M.: A decision framework for integrated wetlandriver basin management in a tropical and data scarce environment, PhD Thesis, ISBN 978-1-138-00025-4, Civil Engineering & Geosciences, TU Delft, Delft, the Netherlands, 188 pp., 2012.

Chiu, L., D-B. Shin and J. Kwaitowski, 2005b: Surface rain rate from TRMM algorithms, in *Earth Science Satellite Remote Sensing*, J. Qu, et al., (Editors), Springer-Verlag (in press)

Chiu, L., Z. Liu, H. Rui, and W. Teng, 2005a: TRMM data and access tools, in *Earth Science Satellite*

Chiu, L., Z. Liu, J. Vongssaard, S. Morain, A. Budge, C. Bales and P. Neville, 2005c: Comparison of TRMM and Water Division rain rates over New Mexico, *Advances in Atmospheric Sci.*, (in press)

Ebert, E.E., 2002: Verifying satellite precipitation estimates for weather and hydrological applications. *1st Intl. Precipitation Working Group (IPWG) Workshop*, Madrid, Spain, 23-27 September 2002.

Huffman G., R. Adler, D. Bolvin and E. Nelkin, 2004: Uncertainty in fine-scale MPA precipitation estimates and implications for hydrometeorological analysis and forecasting, *18th Conf. On Hydrology*, 11-18 January, 2004, Seattle, W.A.

Huffman, G., et al., 2001: Global precipitation at one-degree daily resolution from multisatllite observations, *J. Hydrometeorology*, 2, 36-50.

Huffman, G., R. Adler, B. Rudolf, U. Schneider and P. Keehn, 1995: Global precipitation estimates based on a technique for combining satellite-based estimates, rain gauge analysis and NWP model precipitation information, *J. Clim*., 8, 1284-1295.

Kirtsaeng S., Chantara S. and Kreasuwun J., Mesoscale simulation of a very heavy rainfall event over Mumbai, using the weather research and forecasting (WRF) model. Chaing Mai J. Sci., 2010; 37(3): 429-442.

Kummerow, C. and coauthors, 2000: The status of the Tropical Rainfall Measuring Mission TRMM) sensor package, *J. Atmo. and Ocean Tech.*, 15, 808-816.

Remote Sensing, J. Qu, et al. (Editors), Springer-Verlag (in press)

Shaban A., Robinson C. and El-Baz F., Using MODIS images and TRMM data to correlate rainfall peaks and water discharges from the Lebanese coastal rivers, J. Warp, 2009; 1(4): 227-236.

Thitikon C, Sukrit K : Comparison and Analysis of Remote Sensing-based and Ground-based Precipitation Data Over India, Bureau of Royal Rainmaking and Agricultural Aviation, Bangkok 10900, Thailand.

A STUDY ON VARIATION OF TEMPERATURE AND RAINFALL IN NORTH-CENTRAL REGION OF BANGLADESH

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ABSTRACT

Global climate is one of the most dominating factors related to the well-being of the inhabitants of the earth. Discussions and study on climate change has become a burning issue as it is very much connected to the factors for ensuring human existence on earth. Climatic factors such as temperature and rainfall are closely linked with agricultural production. So knowledge, study and research about the agro-climatic parameters are of utmost importance to formulate the optimum way of resource utilization in order to achieve economic stability in agricultural sector as well as to combat with uncertain climatic phenomenon by developing models. The hydrological region that is North-Central region selected in this study has significant agricultural potential as well as climatic importance in developing models. Mean monthly maximum temperature has been found highest in the district of Faridpur in April and mean monthly minimum temperature has been determined lowest in the district of Tangail in January. Mean monthly rainfall has been found highest in Faridpur district which in July and has been calculated lowest in Tangail district in January.

Keywords*:* *Statistical Analysis, Trend Analysis, Monthly Maximum Temperature, Monthly Minimum Temperature, Monthly Rainfall.*

1. INTRODUCTION

Climate is one of the major driving factors for ensuring the benefits of the inhabitants of the world. Both natural factors as well as anthropogenic causes are responsible for the present changing tendency of global climate. Long-term changes in climatic parameters e.g. temperature and precipitation have direct implications on evaporation demands and consequently agricultural yields. Like the rest of the GBM basin, Bangladesh has a humid subtropical climate. The year can be divided into four seasons: the relatively arid and cool dry season from December to February, the hot and humid premonsoon from March through May, the monsoon from June through September, and the retreating i.e. post monsoon from October to November. The southwest summer monsoon is dominating hydrologic driver in GBM basin. More than 80 percent of annual precipitation occurs during this period. Some areas of the South Asian subcontinent can receive up to 10000 mm of rain. The hydro meteorological characteristics of the three major rivers of GBM basin make the country vulnerable to a range of climate risks, including several flooding and periodic droughts. The performance of agricultural sector is heavily dependent on the annual characteristics of the climatic parameters. The annual patterns of their characteristics have potential impact on agricultural yields, cropping patterns, and productions both from a sectoral perspective as well as in the context of overall economy. Therefore, the threats and challenges faced by agricultural sectors from the climatic factors require continuing research, study and analysis regarding the agro-climatic parameters in order to formulate the optimum way of resource utilization. The hydrological region that is north-central region is selected in this study has significant agricultural potential. In this paper an attempt is made to analyse the climatic data of temperature and rainfall based on statistical approach in the north-central region of Bangladesh.

2. METHODOLOGY

Bangladesh lies between latitudes 20° and 27° north and longitudes 88° and 93° east. Its climate is tropic and humid. Bangladesh has unique hydrological regime. It has been divided into eight hydrological regions classified by Water Resources Planning Organizations (WARPO), Bangladesh. They are –North East(NE), North Central(NC), North West(NW), South East(SE), South Central(SC), South West(SW), Eastern Hilly(EH) and River and Estuary(RE). The North Central hydrological region has been selected as the study area for this research work.

2.1 Profile of the Study Area

The North Central region is bounded by the Jamuna, Padma, Old Brahmaputra and the Sitalakkha river system; Jamuna being on the west, Old Brahmaputra on the north and northeast, Sitalakkha on the east and Padma on the south. Other important waterways are the Dhaleswari-Kaliganga River, which crosses the southwestern part of the region. Besides these main rivers, the region is drained by many small rivers such as, Bangshi, Pungli, and Banar etc.

2.1.1 Detailed Locations of the Stations

There are 4 meteorological stations in North Central region of Bangladesh. Detailed information about the locations of the stations are presented in TABLE 3.1 given below:

Table 1: Locations of Meteorological Stations at North Central Region (Source: BMD)

Name of the Stations	Establishment	Latitude(N) (Degree)	Longitude(E) (Degree)	Elevation(meter)	Station ID
Dhaka	1949	23.767	90.383	8.8	11111
Mymensingh	1883	24.717	90.433	18	10609
Tangail	-	24.25	89.83	10.2	41909
Faridpur	1883	23.6	89.85	8.2	11505

2.2 Data Collection

The data on temperature and rainfall used in this study range from the time period of 1980-2010. The required data were collected from Bangladesh Meteorological Department (BMD) for the four weather stations located in Dhaka, Faridpur, Mymensingh and Tangail in North-Central region of Bangladesh. For the Tangail station the data were available for the period of 1987-2010.

2.3 Methods of Analysis

The collected data of temperature and rainfall have been analysed by using statistical approach i.e. mean, standard deviation and coefficient of variation.

3. RESULTS AND DISCUSSIONS

3.1 Mean Monthly Maximum and Minimum Temperature and Range

From the Figures 1 to 4 and Table 1 it has been found that the mean monthly maximum temperature is occurred highest in the month of April for all the four stations. And the mean monthly minimum temperature is registered lowest in January for all the four stations. And the range of monthly maximum and minimum temperature is highest in January for Dhaka and Mymensingh while it is registered as highest in February for Faridpur and Tangail.

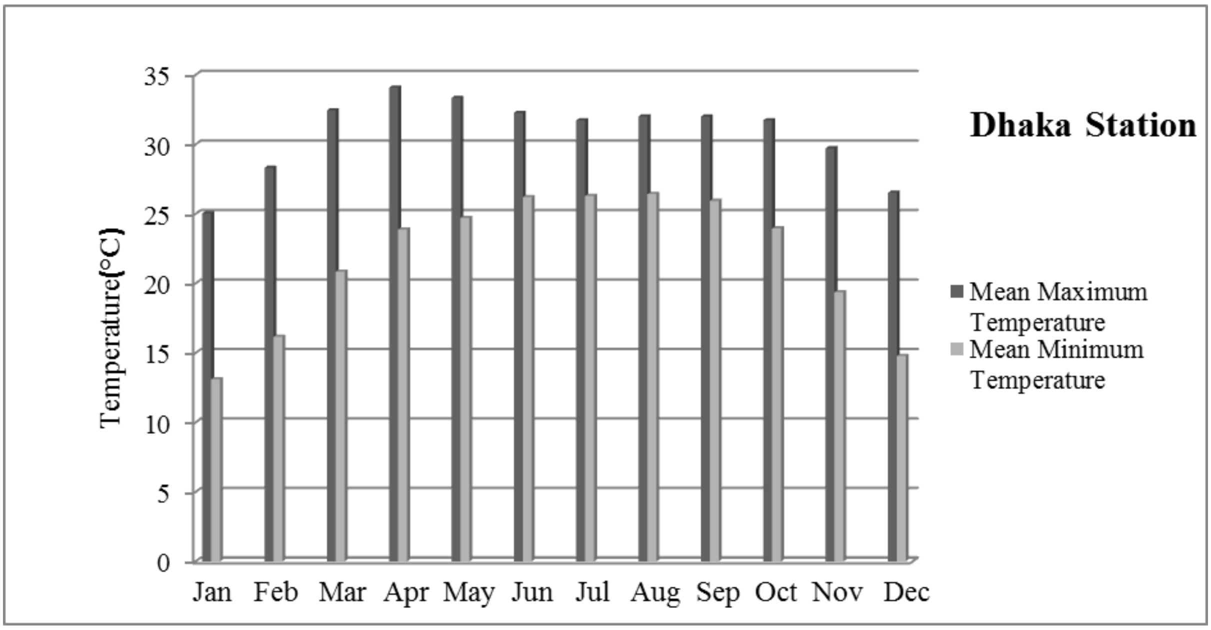

Figure 17: Mean Monthly Maximum and Minimum temperature and in Dhaka Station

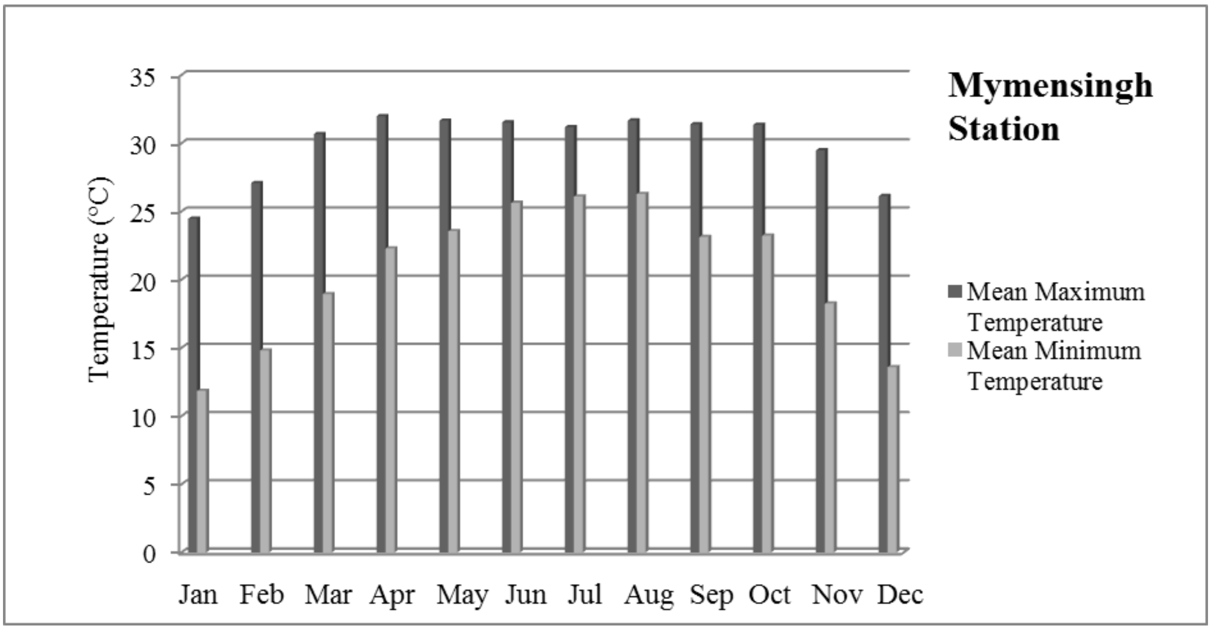

Figure 2: Mean Monthly Maximum and Minimum Temperature in Mymensingh Station

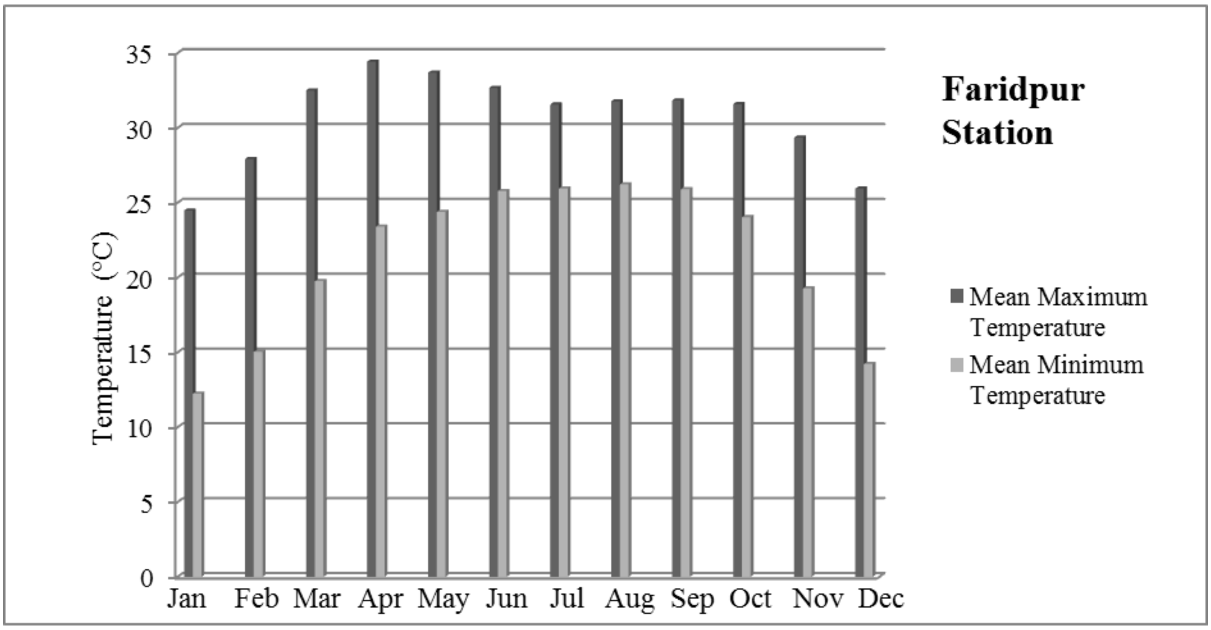

Figure 3: Mean Monthly Maximum and Minimum Temperature in Faridpur Station

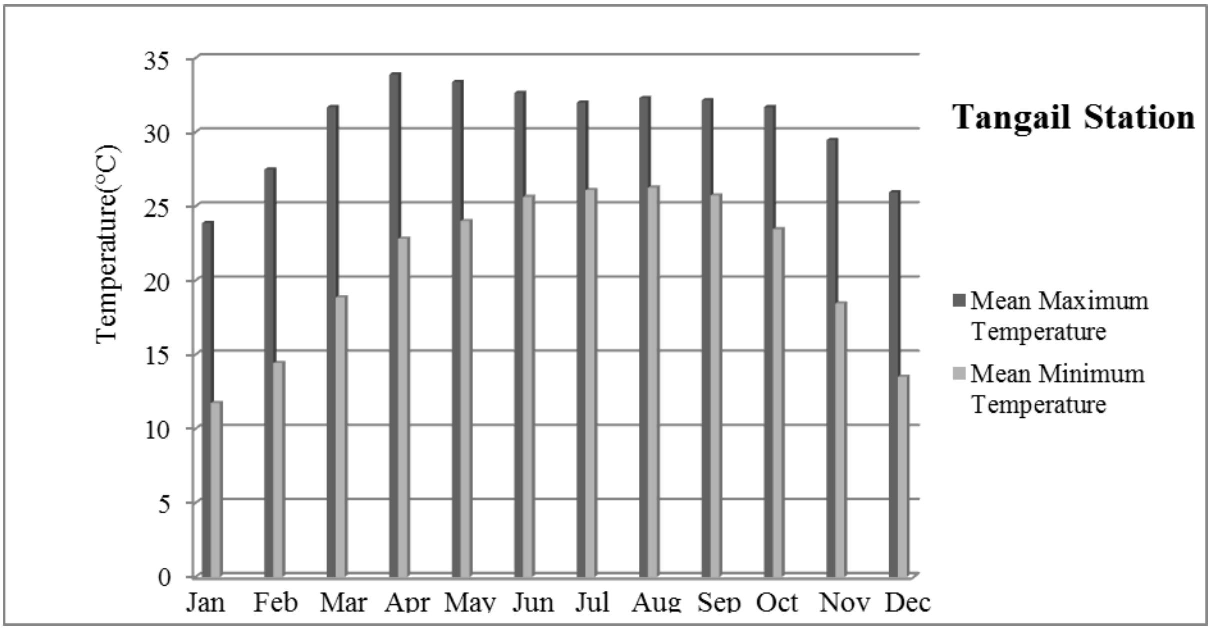

Figure 4: Mean Monthly Maximum and Minimum Temperature in Tangail Station

3.1.1 Equations

The arithmetic mean is the "standard" average, often simply called the "mean".

$$\bar{x} = \frac{1}{n} \cdot \sum_{i=1}^{n} x_i$$

For a data set, the arithmetic mean is equal to the sum of the values divided by the number of values. The arithmetic mean of a set of numbers x1, x2, ..., x_n is typically denoted by pronounced x bar.
For finding mean monthly rainfall and maximum and minimum temperature
Mean = (Sum of Items Value/Nos. of Item)

Standard deviation (δ_{n-1}) is a measure of how widely values are dispersed from the average value. The more is δ_{n-1}, the more dispersed in data and vice versa. The standard deviation is calculated using the "nonbiased" or 'n-1' method. The following formula is used for calculating standard deviation:

$$\delta_{n-1} = \sqrt{\frac{n\sum x^2 - \sum(x)^2}{n(n-1)}}$$

Where, δ_{n-1}= Standard Deviation

x = Variable (Monthly Rainfall or Maximum and Minimum Temperature)

n = nos. of Variables (nos. of year)

The following formula was used for calculating coefficient of variation:

$$C_V = \frac{\delta_{n-1}}{\bar{x}}$$

Where C_V = Coefficient of Variation

$\bar{x}$ = mean of monthly rainfall or maximum or minimum temperature

3.1.2 Tables

Table 1: Mean Monthly Maximum and Minimum Temperature and Range

Station Name		Temperature(°C)											
		Jan	Feb	Mar	Apr	May	Jun	July	Aug	Sep	Oct	Nov	Dec
Dhaka	Max	25.03	28.1	32.4	34.0	33.34	32.2	31.7	32.0	31.9	31.7	29.7	26.5
	Min	13.10	16.1	20.8	23.8	24.7	26.2	26.2	26.4	25.9	23.9	19.3	14.7
	Range	11.93	12	11.6	10.2	8.64	6	5.5	5.6	6	7.8	10.4	11.8
Mymensingh	Max	24.46	27.0	30.6	31.9	31.65	31.5	31.2	31.7	31.4	31.3	29.5	26.1
	Min	11.85	14.8	18.9	22.3	23.55	25.6	26.1	26.3	23.1	23.2	18.2	13.6
	Range	12.71	12.2	11.7	9.6	8.1	5.9	5.1	5.4	8.3	8.1	11.3	12.5
Faridpur	Max	24.47	27.9	32.5	34.4	33.7	32.7	31.6	31.8	31.8	31.6	29.4	25.9
	Min	12.25	15.0	19.8	23.4	24.4	25.8	25.9	26.2	25.8	24.0	19.3	14.2
	Range	12.22	12.9	12.7	11	9.3	6.9	5.7	5.6	6	7.6	10.1	11.7
Tangail	Max	23.9	27.5	31.7	33.9	33.4	32.6	32	32.3	32.1	31.7	29.5	25.9
	Min	11.7	14.4	18.9	22.8	24	25.6	26	26.3	25.7	23.5	18.4	13.5
	Range	12.2	13.1	12.8	11.1	9.4	7.1	6	6	6.4	8.2	11.1	12.4

Station Name		Temperature(°C)											
		Jan	Feb	Mar	Apr	May	Jun	July	Aug	Sep	Oct	Nov	Dec
Dhaka	STDEV	1.16	1.59	1.54	1.45	1.14	1.55	0.56	0.59	0.69	0.74	0.63	0.82
	CV	0.05	0.06	0.05	0.04	0.03	0.05	0.02	0.02	0.02	0.02	0.02	0.03
Mymensingh	STDEV	1.13	1.34	1.39	1.46	1.00	1.06	0.5	0.6	0.6	0.7	0.74	0.78
	CV	0.05	0.05	0.04	0.04	0.03	0.03	0.02	0.02	0.02	0.02	0.03	0.03
Faridpur	STDEV	1.04	1.23	1.19	1.38	1.22	0.66	0.55	0.55	0.82	0.67	0.55	0.84
	CV	0.04	0.05	0.04	0.04	0.04	0.02	0.02	0.02	0.03	0.03	0.02	0.03
Tangail	STDEV	1.09	1.43	1.32	1.49	1.12	0.84	0.62	0.57	0.68	0.77	0.46	0.87
	CV	0.05	0.05	0.04	0.04	0.03	0.03	0.02	0.02	0.02	0.02	0.02	0.03

Table 2: Standard Deviation and Coefficient of Variation of Monthly Maximum Temperature

Table 3: Standard deviation and Coefficient of Variation of Monthly Minimum Temperature

Station Name		Temperature(°C)											
		Jan	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**July**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
Dhaka	**STDEV**	1	1.12	1.2	1.22	0.87	0.61	0.45	0.38	0.48	0.74	0.87	1.08
	CV	0.08	0.07	0.06	0.05	0.04	0.02	0.02	0.01	0.02	0.03	0.05	0.07
Mymensingh	**STDEV**	0.99	1.2	1.2	1.15	0.87	0.56	0.52	0.52	0.59	0.7	0.95	0.98
	CV	0.08	0.08	0.06	0.05	0.04	0.02	0.02	0.02	0.02	0.03	0.05	0.07
Faridpur	**STDEV**	0.91	1.24	1.08	1.02	1.52	0.52	0.49	0.45	0.8	0.87	0.98	0.96
	CV	0.07	0.08	0.06	0.05	0.06	0.02	0.02	0.02	0.03	0.04	0.05	0.07
Tangail	**STDEV**	2	1.04	1.16	1.05	0.74	0.47	0.28	0.28	0.37	0.71	0.8	0.92
	CV	0.17	0.07	0.06	0.05	0.03	0.02	0.01	0.01	0.01	0.03	0.04	0.07

From the Table 2 and 3 it has been observed that the data points are very closely near to the mean (Standard deviation <1) or slightly deviated from the mean (Standard Deviation >1) for both monthly maximum and monthly minimum temperature. From the observation of the values of coefficient of variations it has been observed that the amount of variation of data sets ranges from 0.02 to 0.06 for monthly maximum temperature and it ranges from 0.01 to 0.08 for monthly minimum temperature.

3.2 Mean, Standard Deviation and Coefficient of Variation of Monthly Rainfall

From the interpretation of the Table 4 and Figure 5 to 8 it has been found that mean monthly rainfall is lowest in magnitude for the month of January and highest in magnitude at the month of July in this respective region.

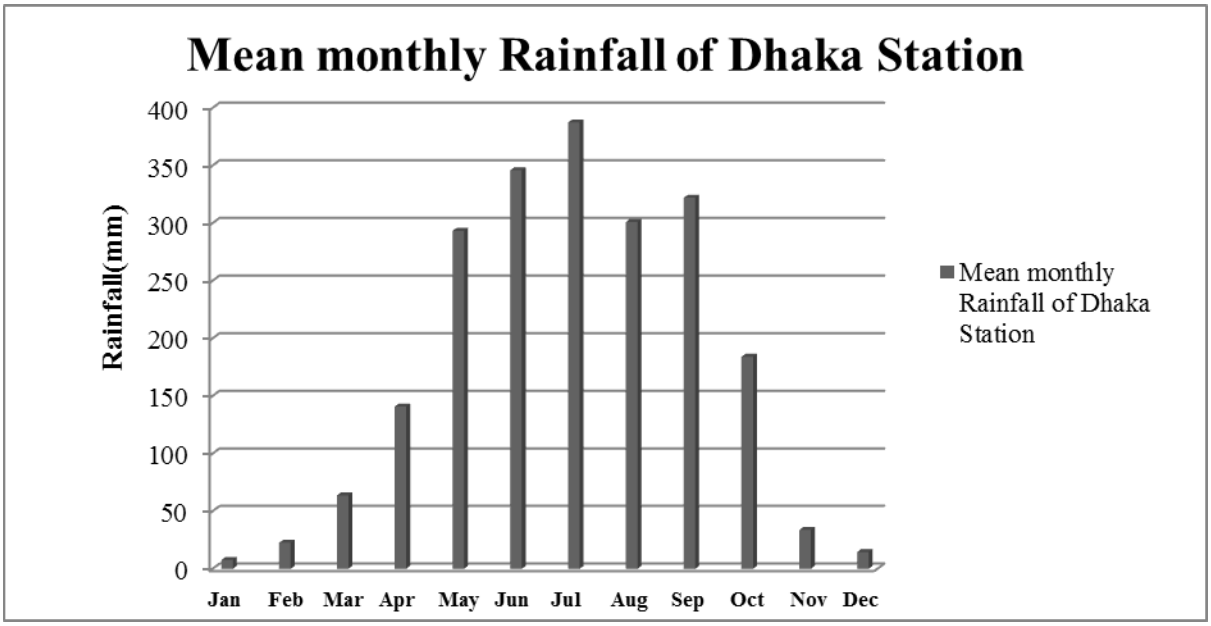

Figure 5: Mean Monthly Rainfall in Dhaka Station

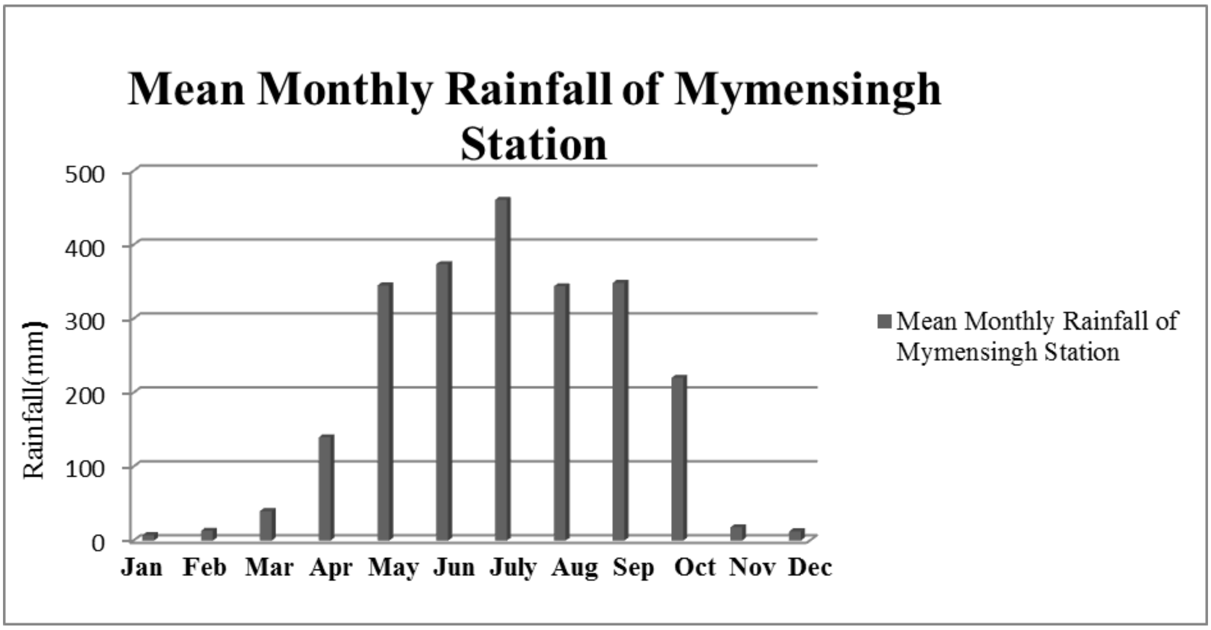

Figure 6: Mean Monthly Rainfall in Mymensingh Station

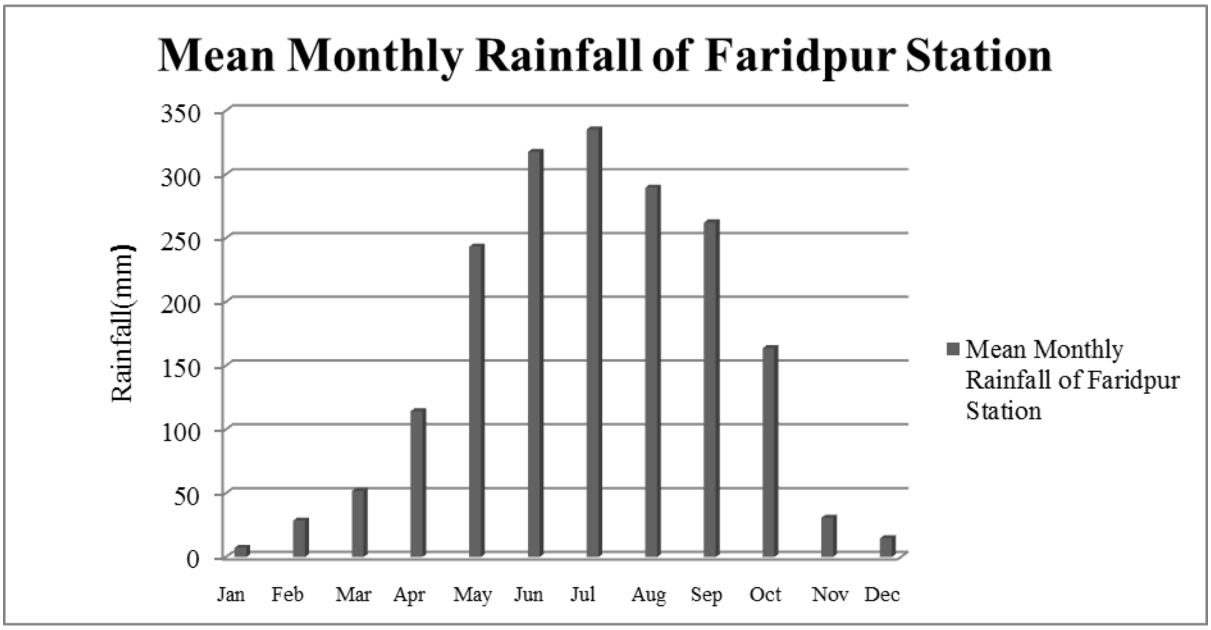

Figure 7: Mean Monthly Rainfall in Faridpur Station

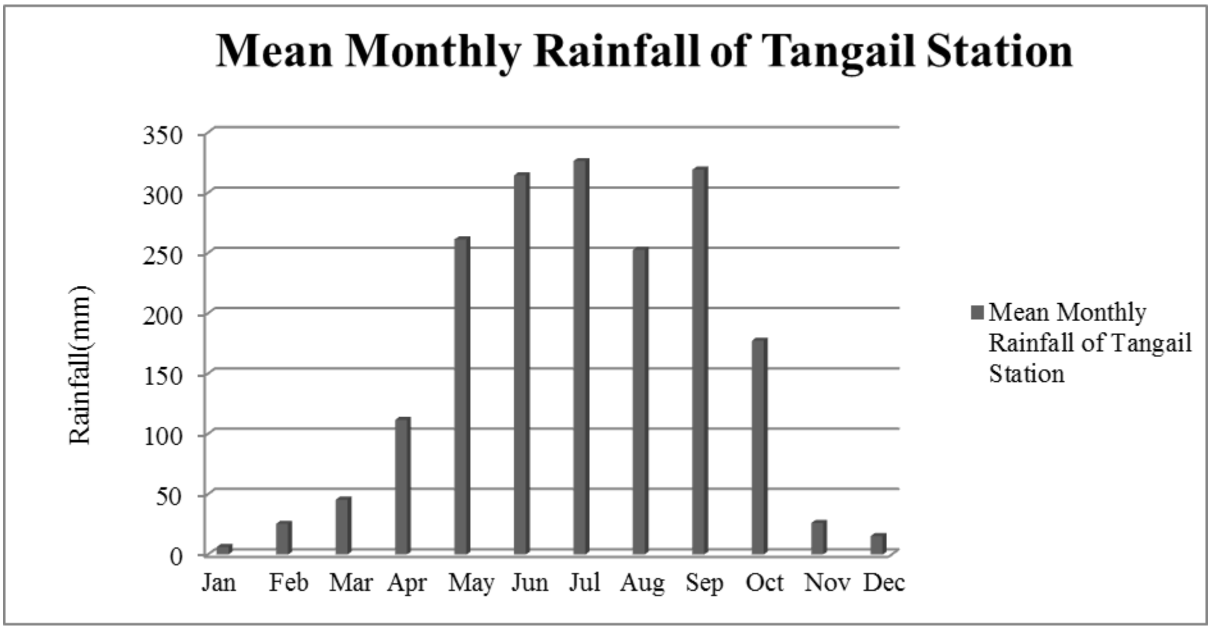

Figure 8: Mean Monthly Rainfall in Tangail Station

Station Name		Rainfall(mm)											
		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Dhaka	MEAN	7.54	22.6	63.7	140.8	293.3	345.9	387.3	300.9	321.9	184	33.68	14.42
	STDV	11.11	20.8	55.03	78.16	146.5	141.5	166.9	113.6	150.2	110.1	51.25	32.94
	CV	1.47	0.92	0.86	0.56	0.5	0.41	0.43	0.38	0.47	0.6	1.52	2.28
Mymensingh	MEAN	7.32	28.7	51.89	114.4	243.4	317.7	335.4	289.7	262.4	163.8	30.82	14.8
	STDV	11.95	29.9	52.55	71.55	102.1	131.5	115.3	133.5	174.6	110.7	44.96	32.67
	CV	1.63	1.04	1.01	0.63	0.42	0.41	0.34	0.46	0.67	0.68	1.46	2.21
Faridpur	MEAN	7.16	13.2	39.85	139.5	344.9	373.9	460.8	343.9	348.5	220.1	17.8	12.37
	STDV	13.06	19.0	34.8	89.15	164.6	212.9	182.8	145.6	175.7	148.8	34.01	28.32
	CV	1.82	1.45	0.87	0.64	0.48	0.57	0.4	0.42	0.5	0.68	1.91	2.29
Tangail	MEAN	6.3	25.5	45.5	111.7	261.3	314.4	326.3	252.7	319.2	177.4	26.17	15.33
	STDV	8.61	22.4	46.7	74.7	100.7	141.4	131.9	99.5	229.7	125.3	36.5	33.64
	CV	1.37	0.89	1.02	0.67	0.39	0.45	0.4	0.39	0.72	0.71	1.39	2.19

Table 4: Mean, Standard Deviation and Coefficient of Variation of Monthly Rainfall

From Table 4 it has been observed that the data points are very closely near to the mean (Standard deviation <100) or slightly deviated from the mean (Standard Deviation >100) for mean monthly rainfall. From the observation of the values of coefficient of variations it has been observed that the amount of variation of data sets ranges from 0.4 to 2.29 for mean monthly rainfall in this respective region.

4. CONCLUSIONS

The conclusions extracted from this research paper can be stated as follows:

- The mean monthly maximum temperature remains very high in magnitude during the period April-May and the mean monthly minimum temperature remains very low in magnitude during the period December-January and the monthly variation of maximum and minimum temperature is very high during the period January to March and very low during the period June to September.
- The mean monthly maximum temperature remains highest in Faridpur in April which is 34.4° C in magnitude and the mean monthly minimum temperature remains lowest in Tangail in January which is 11.7°C in magnitude.
- The mean monthly rainfall remains very high in magnitude during the period May to September and remains very low in magnitude during the period December to February in north-central region.
- The mean monthly rainfall remains highest in Faridpur district which is 460.8 mm in magnitude in July and remins lowest in Tangail District which is 6.3 mm in magnitude in January.

REFERENCES

Al-Muyeed, A. and Shadullah, A.M. (2010). “A Study on Variation of Temperature and Precipitation of Bangladesh”, THE AUST Journal of Science and Technology.

Ayub, R. (2011). “Trend Analysis of Climatic Parameters and Effect on Their Change on Reference Crop Evaporation (ETO) in North West Region of Bangladesh”, B.Sc. in Engineering Thesis, Department of Water Resources Engineering, BUET, Dhaka.

Islam, A.K.M.S. (2008). “Analyzing Changes of Temperature over Bangladesh due to Global Warming using Historical Data”, IWFM, BUET, Dhaka.

Manalo, E.B. (1976). “Agro-Climatic Survey of Bangladesh”, Bangladesh Rice Research Institute, Dhaka.

Mitu, M.F. (2012). “Assessment of Agro-Climatic Parameters in North West Region of Bangladesh”, B.Sc. in Engineering Thesis, Department of Water Resources Engineering, BUET, Dhaka.

Rahman, M.S. (2002). “A Study on Agro-Climatic Parameters on Bangladesh”, M. Engineering Project Report, Department of Water Resources Engineering, BUET,Dhaka.

STUDY OF PRECIPITATION VARIABILITY IN BANGLADESH: AN INDEX BASED APPROACH

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ABSTRACT

Drought, which is one the major issues of today's world have several inverse impacts on economy, society and ecology. The severity of the drought is related to main usage of water and can be expressed in different indices. The SPI and RAI are the most widely used drought indices to provide good estimations about the intensity, magnitude and spatial extent of droughts. In this study, the SPI and RAI are calculated for 12-months time scale based on their mathematical expression. Based on the SPI and RAI values, the intensity of drought is estimated and compared for two equal periods of 20 years each (1968~1987 and 1988~2007) in different locations of Bangladesh. SPI and RAI patterns were found to be changed due to change of time scales for available rainfall data. It is observed that SPI is to a great extent more suitable for describing meteorological drought in the tropical environment and is more efficient in detecting mild drought than RAI. It is also found that the total number of drought years have increased from period 1 (1968~1987) to period 2 (1988~2007) for different locations of Bangladesh.

Keywords: *Precipitation Variability, Meteorological Drought, RAI Index, SPI Index*

1. INTRODUCTION

Droughts are one of the world's most severe and collectively affective natural disasters that cause an average $6-8 billion in global damages yearly (Wilhite, 2000). Drought is not a recent phenomenon in Bangladesh. Historical record indicates that drought has occurred frequently in the past. Some of these droughts were severe and accompanied by famines, arising from crop failures. The usable water sources consist of soil moisture, groundwater, snow pack, runoff and reservoir storage. Any drought is directly related with the one or more of these five sources of supply. The time scale over which precipitation deficits accumulate becomes extremely important and functionally separates different types of drought. Agricultural (soil moisture) droughts, for example, typically have a much shorter time scale than hydrologic (groundwater, runoff and reservoir storage) droughts (Mckee et al, 1993).

Several different indices have been developed to define types of droughts. These indices have been presented in different studies (Keyantash and Dracup, 2002) where most of them Standardized Precipitation Index (SPI), Rainfall Anomaly Index (RAI), Standardized Rainfall Index (SRI), Surface Humidity Index (SHI)) are separate indices and define one of the previously defined drought types. Among the indices SPI and RAI are the most used ones for defining the meteorological drought by using the precipitation data. In this study, the analysis of drought events has been carried out by applying the SPI and the RAI. The meteorological rainfall data of 34 rain gauge stations is collected from Bangladesh Agricultural Research Council (BARC) and the data is used for two equal periods of 20 years each (1968-1987 and 1988-2007).

The Standardized Precipitation Index (SPI) is a tool which was developed primarily for defining and monitoring drought. It allows an analyst to determine the rarity of a drought at a given time scale (temporal resolution) of interest for any rainfall station with historic data. It can also be used to determine periods of anomalously wet events. The SPI is not a drought prediction tool. Hayes (2000) demonstrated how the SPI at varying time scales could have been used operationally to monitor the 1996 drought from its development to its conclusion in the southern Great Plains and the south-western USA. They concluded that 'using the SPI as a drought monitoring tool will improve the timely identification of emerging drought conditions that can trigger appropriate state and federal actions'. The length of record for precipitation used in the SPI calculation is 'ideally a continuous period of at least 30 years' (Mckee et al., 1993). Unfortunately, the 30 year record requirement cannot be met in some developing countries. Even in the USA, the precipitation record length varies from one station to another across the region of interest. In Nebraska, for instance, among the 201 Cooperative Observer Program (COOP) stations with precipitation records that spans at least 30 years, the beginning years of precipitation record vary from 1880 to1971. Because the SPI is a probability-related index, the longer the length of record the more confidence there is in the stability of the underlying statistics. The **Standardized Precipitation Index (SPI)** (McKee et al.1993) can be defined on each of the time scales as the difference between precipitation on the time series (x_i) and the mean value ($\overline{x}$) divided by the standard deviation (s) as,

$$SPI = \frac{x_i - \overline{x}}{s} \tag{1}$$

Table 1: Drought categorization values for SPI (Mckee et al., 1993)

SPI Values	Drought Category
0 to -0.99	Mild Drought
-1.00 to -1.49	Moderate Drought
-1.50 to -1.99	Severe Drought

Table 1 shows the drought category for different range of SPI values. The **12** month SPI is a comparison of the precipitation for 12 consecutive months during all the previous years of available data. The SPI at this time scale reflects long-term precipitation patterns. In some locations of the country, the 12-month SPI is most closely related with the Palmer Index and the two indices should reflect similar conditions.

The **Rainfall Anomaly Index (RAI)** was developed by Van Rooy (1965) to incorporate a ranking procedure to assign magnitudes to positive and negative precipitation anomalies. Oladipo (1985) reported that the index values should be judged against a 9-member classification scheme, ranging from extremely wet to extremely dry. They found that differences between the RAI and the more complicated indices of Palmer and Bhalme-Mooley were negligible. The RAI can be defined as,

$$RAI = \pm 3 \frac{P_i - \overline{P}}{\overline{E} - \overline{P}} \tag{2}$$

Where P_i is annual precipitation, $\overline{P}$ is average annual precipitation for given period and $\overline{E}$ is average of ten extremes for the investigated period. For positive anomalies, the prefix is positive and $\overline{E}$ is the average of the 10 highest precipitation values on record; for negative anomalies, the prefix is negative and the 10 lowest measurements are used. Table 2 shows the drought category for different range of RAI values.

Table 2: Drought categorization values for RAI (Ma'aruf et al. 2011)

RAI Values	Drought Category
-1.00 to -1.99	Mild Drought
-2.00 to -2.99	Moderate Drought
-3.00 or less	Severe Drought

Bangladesh is primarily a low-lying plain of about 144000 km^2, situated on deltas of large rivers flowing from the Himalayas. Geographically, it extends from $20^0 34'$ N to $26^0 38'$ N latitude and from $88^0 01'$ E to $92^0 41'$ E longitude. Bangladesh has a sub-tropical humid climate characterized by wide seasonal variations in rainfall, moderately warm temperatures and high humidity. Bangladesh is affected by major country-wide droughts every five years. However, local droughts occur frequently and affect crop life cycles. The agricultural drought, related to soil moisture deficiency, occurs at various stages of crop growth. Monsoon failure often brings yield reduction and famine to the affected regions. A better understanding of the monsoon cycle is clearly of major scientific and social value between 1960 and 1991, droughts occurred in Bangladesh 19 times. Very severe droughts hit the country in 1951, 1961, 1975, 1979, 1981, 1982, 1984, 1989, 1994, 1995 and 2000. Past droughts have typically affected about 47 percent of the country and 53 percent of the population. The associated decline in crop production, losses of assets and lower employment opportunities contributed to increased household food insecurity. Food consumption fell, along with household ability to meet food needs on a sustainable basis. Vegetables and many other pulses are in short supply during drought. In this study, the SPI and RAI are calculated for 12-months time scale based on their mathematical expression. Based on the SPI and RAI values, the intensity of drought is estimated and compared for two equal periods of 20 years each (1968~1987 and 1988~2007) in different locations of Bangladesh.

2. METHODOLOGY

2.1 Study Area and Data Collection

The rainfall data of 34 different rain gauge stations from different location of Bangladesh has been collected for the SPI analysis. Figure 3.1 shows the map for the locations of rain gauge stations. The data ranges were varied from the year 1968 to 2007· Table 3.1 shows the division wise list of rain gauge stations with data ranges, for which the monthly rainfall records are available.

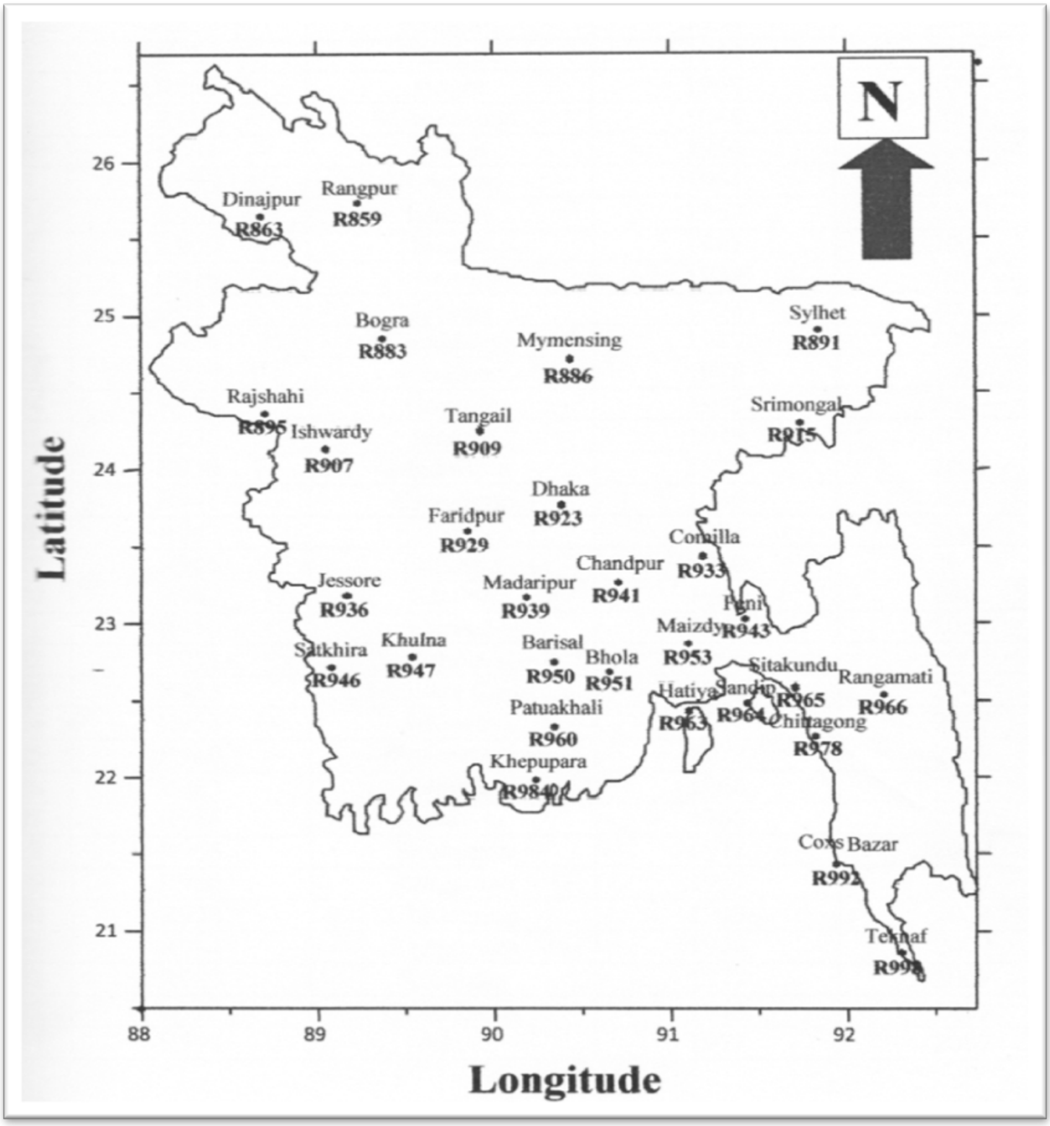

Figure 18: Map showing the locations of rain gauge station

Table 3: Details of rain gauge stations with available data ranges

Division	SI No.	Station Name	Station Number	Data Ranges
Rajshahi	1	Rajshahi	R895	1968-2007
	2	Bogra	R883	1968-2007
	3	Ishurdy	R907	1968-2007
Khulna	4	Khulna	R947	1968-2007
	5	Satkhira	R946	1968-2007
	6	Mongla	R958	1991-2007
	7	Chuadanga	R926	1989-2007
	8	Jossore	R936	1968-2007
Dhaka	9	Dhaka	R923	1968-2007
	10	Tangail	R909	1987-2007
	11	Madaripur	R939	1977-2007
	12	Mymensingh	R886	1968-2007
	13	Faridpur	R929	1968-2007
Chittagong	14	Chittagong	R978	1968-2007
	15	Rangamati	R966	1968-2007
	16	Cox's Bazar	R992	1968-2007
	17	Teknaf	R998	1977-2007
	18	Sitakunda	R965	1977-2007
	19	Feni	R943	1973-2007
	20	Kutubdia	R989	1977-2007
	21	Comilla	R933	1968-2007
	22	Chandpur	R941	1968-2007
	23	Hatiya	R963	1968-2007
	24	Sandwip	R964	1968-2007
Barisal	25	Khepupara	R984	1974-2007
	26	Barisal	R950	1948-2007
	27	Bhola	R951	1968-2007
	28	Maijdee Court	R953	1968-2007
	29	Patuakhali	R960	1973-2007
Sylhet	30	Sylhet	R891	1968-2007
	31	Srimongal	R915	1968-2007
Rangpur	32	Sayedpur	R858	1991-2007
	33	Rangpur	R859	1968-2007
	34	Dinajpur	R863	1968-2007

2.2 SPI Calculation

The rainfall data of 34 different rain gauge stations from different location of Bangladesh has been selected for the SPI analysis. The data ranges were varied from the year 1968 to 2007. Total monthly rainfalls for all stations are calculated from the daily rainfall data. From monthly precipitation data, the SPI was calculated by using the software 'SPI_SL_6' of National Drought Mitigation Center, Zimbabwe for 12-month time scales. From the graphical representation of SPI, the intensity of drought category is calculated for two equal periods of 20 years each (1968-1987 and 1988-2007) in different locations of Bangladesh.

The SPI for various seasons in Bangladesh was calculated by using the following equation,

$$SPI = \frac{x_i - \bar{x}}{s}$$

2.3 RAI Calculation

From monthly precipitation data, the RAI was calculated by using the following equation,

$$RAI = \pm 3\frac{P_i - \bar{P}}{\bar{E} - \bar{P}}$$

From the graphical representation of RAI, the intensity of drought category is calculated for two equal periods of 20 years each (1968-1987 and 1988-2007) in different locations of Bangladesh.

3. RESULTS AND DISCUSSION

The monthly rainfall data for 34 rain gauge stations are used for SPI and RAI analysis. The data ranges were varied from the year 1968 to 2007. The whole periods for 23 stations are divided in two parts and SPI and RAI were calculated for period 1 (1968~1987) and period 2 (1988~2007) separately in different locations of Bangladesh.

3.1 SPI and RAI for Rainfall Data (Period 1 & Period 2)

Fig. 2 and Fig. 3 show the 12-months SPI and RAI pattern for the Rajshahi for precipitation data 1968-1987. Fig. 4 and Fig. 5 show the 12-months SPI and RAI pattern for the Rajshahi for data 1988-2007. It is observed that SPI and RAI show similar pattern of index variation.

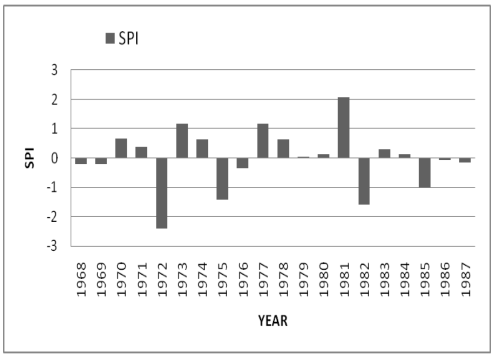

Figure 2: Variation of SPI for period 1

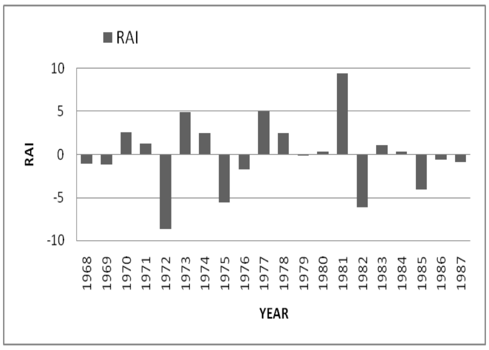

Figure 3: Variation of RAI for period 1

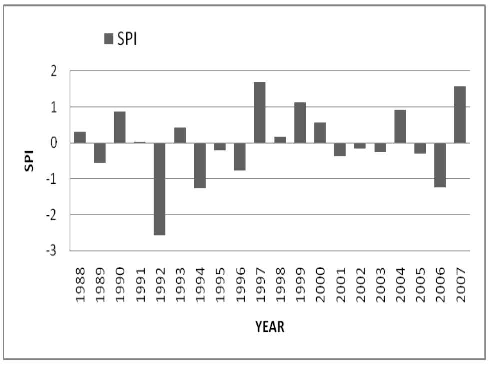

Figure 4: Variation of SPI for period 2

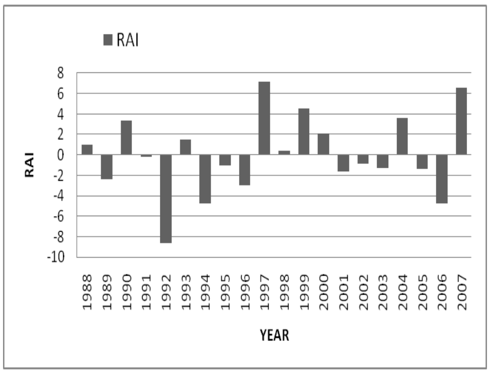

Figure 5: Variation of SPI for period 2

3.2 Comparison Between SPI and RAI during Two periods

The analysis of RAI and SPI for the rainfall data reveals that the zone (Rajshahi) has experienced mild, moderate and severe drought during the two study periods (Tables 4 and 5).

Table 4: Drought events during period 1(1968-1987)

Intensity	Drought years based on SPI	Drought years based on RAI
Mild drought	1968,1969,1976,1986,1987	1968,1969,1976
Moderate drought	1975,1985	——
Severe drought	1972,1982	1972,1975,1982,1985

Table 5: Drought events during period 2 (1988-2007)

Intensity	Drought years based on SPI	Drought years based on RAI
Mild drought	1989,1995,1996,2001,2002,2003,2005	1995,2001,2003,2005
Moderate drought	1994,2006	1989,1996
Severe drought	1992	1992,1994,2006

Table 4 shows that the RAI was able to detect 3 years of mild drought in Rajshahi while the SPI detected 5 years of mild drought in 1968-1987 periods. Table 5 shows that the SPI detected 7 years of mild drought whereas the RAI detected only 4 years. There is also variation in number of moderate and severe droughts. The drought years for other stations are provided in Table 6.

Table 6: Drought years for two periods

<table>
<tr><th rowspan="2">Station</th><th rowspan="2">Period</th><th rowspan="2">Drought Category</th><th colspan="2">Drought years</th><th colspan="2">Drought years</th></tr>
<tr><th>SPI</th><th>No. of years</th><th>RAI</th><th>No. of years</th></tr>
<tr><td rowspan="6">Bogra</td><td rowspan="3">Period 1</td><td>Mild</td><td>1968,1969,1980,1981,1985</td><td>5</td><td>1968,1980</td><td>2</td></tr>
<tr><td>Moderate</td><td>1975</td><td>1</td><td>1969</td><td>1</td></tr>
<tr><td>Severe</td><td>1971,1972,1982</td><td>3</td><td>1971,1972,1975,1985</td><td>4</td></tr>
<tr><td rowspan="3">Period 2</td><td>Mild</td><td>1989,1996,1999,2004</td><td>4</td><td>1996</td><td>1</td></tr>
<tr><td>Moderate</td><td>1992,1994,1997,2001,2002,2007</td><td>6</td><td>1989,1999,2001,2002</td><td>4</td></tr>
<tr><td>Severe</td><td>---</td><td>0</td><td>1992,1994,1997,2007</td><td>4</td></tr>
<tr><td rowspan="6">Ishurdy</td><td rowspan="3">Period 1</td><td>Mild</td><td>1968,1974,1979,1980,1986,1987</td><td>6</td><td>1968,1969,1980,1986</td><td>4</td></tr>
<tr><td>Moderate</td><td>1971,1978,1982,1985</td><td>4</td><td>---</td><td>0</td></tr>
<tr><td>Severe</td><td>1972</td><td>1</td><td>1971,1972,1978,1982,1985</td><td>5</td></tr>
<tr><td rowspan="3">Period 2</td><td>Mild</td><td>1989,1995,1996,2001,2002,2006</td><td>6</td><td>1996</td><td>1</td></tr>
<tr><td>Moderate</td><td>---</td><td>0</td><td>1989,1995,2006</td><td>3</td></tr>
<tr><td>Severe</td><td>1992,1994,2003</td><td>3</td><td>1992,1994,2003</td><td>3</td></tr>
<tr><td rowspan="6">Khulna</td><td rowspan="3">Period 1</td><td>Mild</td><td>1968,1975,1977,1982,1985</td><td>5</td><td>1975,1982,1985</td><td>3</td></tr>
<tr><td>Moderate</td><td>1973</td><td>1</td><td>1968,1977</td><td>2</td></tr>
<tr><td>Severe</td><td>1971,1972</td><td>2</td><td>1971,1972,1973</td><td>3</td></tr>
<tr><td rowspan="3">Period 2</td><td>Mild</td><td>1991,1999,2000,2001,2003</td><td>5</td><td>1999,2000,2001</td><td>3</td></tr>
<tr><td>Moderate</td><td>1989,1996</td><td>2</td><td>2003</td><td>1</td></tr>
<tr><td>Severe</td><td>1994</td><td>1</td><td>1989,1994,1996</td><td>3</td></tr>
<tr><td rowspan="6">Satkhira</td><td rowspan="3">Period 1</td><td>Mild</td><td>1976,1982,1985</td><td>3</td><td>1985</td><td>1</td></tr>
<tr><td>Moderate</td><td>1979,1980</td><td>2</td><td>1976,1982</td><td>3</td></tr>
<tr><td>Severe</td><td>1972</td><td>1</td><td>1979,1980</td><td>3</td></tr>
<tr><td rowspan="3">Period 2</td><td>Mild</td><td>1991,1995,1998,2001,2003</td><td>5</td><td>1998,2003</td><td>2</td></tr>
<tr><td>Moderate</td><td>1994,1996,1999</td><td>3</td><td>2001</td><td>1</td></tr>
<tr><td>Severe</td><td>1989,1999</td><td>2</td><td>1989,1992,1994,1996,1999</td><td>5</td></tr>
<tr><td rowspan="6">Jessore</td><td rowspan="3">Period 1</td><td>Mild</td><td>1971,1980</td><td>2</td><td>1971</td><td>1</td></tr>
<tr><td>Moderate</td><td>1975,1976,1982,1985</td><td>4</td><td>1980</td><td>1</td></tr>
<tr><td>Severe</td><td>1972</td><td>1</td><td>1972,1975,1976,1982,1985</td><td>5</td></tr>
<tr><td rowspan="3">Period 2</td><td>Mild</td><td>1995,1997,1998,1999,2001,2005</td><td>6</td><td>1997</td><td>1</td></tr>
<tr><td>Moderate</td><td>1992</td><td>1</td><td>1995,1998,1999,2001</td><td>4</td></tr>
<tr><td>Severe</td><td>1989,1994</td><td>2</td><td>1989,1992,1994</td><td>3</td></tr>
<tr><td rowspan="6">Dhaka</td><td rowspan="3">Period 1</td><td>Mild</td><td>1968,1970,1974,1977,1981,1985</td><td>6</td><td>1970,1974</td><td>2</td></tr>
<tr><td>Moderate</td><td>1969,1972,1979,1982</td><td>4</td><td>1968,1977,1979,1981</td><td>4</td></tr>
<tr><td>Severe</td><td>---</td><td>0</td><td>1969,1972,1982</td><td>3</td></tr>
<tr><td rowspan="3">Period 2</td><td>Mild</td><td>1995,1996,1997,2002,2006</td><td>5</td><td>1997,2002,2006</td><td>3</td></tr>
<tr><td>Moderate</td><td>1989,1994,2001,2003</td><td>4</td><td>1995</td><td>1</td></tr>
<tr><td>Severe</td><td>1992</td><td>1</td><td>1989,1992,1994,2001,2003</td><td>5</td></tr>
</table>

Mymensingh	Period 1	Mild	1985	1	1985	1
		Moderate	1973,1974,1979	3	---	0
		Severe	1975	1	1973,1974,1975,1979	4
	Period 2	Mild	1997,1999,2001,2006	4	---	0
		Moderate	1992,1994,1996,2003	4	2001,2006	2
		Severe	1991	1	1991,1992,1994,1996,2003	5
Faridpur	Period 1	Mild	1968,1972,1979	3	1979	1
		Moderate	1969,1971,1982,1985	4	1968,1972	2
		Severe	1978	1	1969,1971,1978,1982,1985	5
	Period 2	Mild	1997,2001,2005,2006	4	2005	1
		Moderate	1989,2003	2	1997,2001,2006	3
		Severe	1992,1994	2	1989,1992,1994,2003	4
Chittagong	Period 1	Mild	1968,1975,1977,1978,1981,1984	6	1968,1975,1984	3
		Moderate	1976	1	1977,1978,1981	3
		Severe	1972	1	1972,1976	2
	Period 2	Mild	1989,2002,2003	3	2002,2003	2
		Moderate	1992,1994,1995,2001,2005,2006	6	1989,1995	2
		Severe	---	0	1992,1994,2001,2005,2006	5
Rangamati	Period 1	Mild	1971,1975,1981,1984	4	1975,1984	2
		Moderate	1974,1985	2	1971,1981	2
		Severe	1972,1982	2	1972,1974,1982,1985	4
	Period 2	Mild	1989,1990,1995,2003,2006	5	1990,2003	2
		Moderate	2001,2005	2	1995,2006	2
		Severe	1992,1994	2	1992,1994,2001,2005	4
Cox's Bazar	Period 1	Mild	1976,1981,1986	3	1981,1986	2
		Moderate	1972	1	1976	1
		Severe	1979,1980	2	1972,1979,1980	3
	Period 2	Mild	1993,1994,1995,1996,1997	5	---	0
		Moderate	2004,2005	2	1993,1994,1995,1997	4
		Severe	1989,1992	2	1989,1992,2004,2005	4
Comilla	Period 1	Mild	1970,1975,1976,1987	4	1987	1
		Moderate	1972,1979,1985	3	1970,1975	2
		Severe	1971	1	1971,1972,1979,1985	4
	Period 2	Mild	1996,2000,2001,2003,2006	5	1996,2001	2
		Moderate	---	0	2003,2006	2
		Severe	1989,1992,1994	3	1989,1992,1994	3
Chandpur	Period 1	Mild	1970,1972,1973,1978	4	1970,1972,1973	3
		Moderate	1971,1975	2	1978	1
		Severe	1976,1977	2	1971,1975,1976,1977	4
	Period 2	Mild	1996,1997,1998,2000,2003,2006	6	1996,1998,2000	3
		Moderate	1989,1994,2005	3	2003,2006	2
		Severe	1992	1	1989,1992,1994,2005	4
Hatiya	Period 1	Mild	1968,1970,1985,1986,1987	5	1985,1987	2
		Moderate	1971,1982	2	1970,1986	2
		Severe	1972,1973	2	1971,1972,1973,1982	4
	Period 2	Mild	1989,1992,1996,1997	4	1992,1997	2
		Moderate	1990,1994	2	1989,1996	2
		Severe	1993	1	1990,1993,1994	3
Sandwip	Period 1	Mild	1968,1969,1970,1986,1987	5	---	0
		Moderate	1978	1	1968,1969,1970,1986	4
		Severe	1975,1979,1984	3	1975,1978,1979,1984	4
	Period 2	Mild	1991,1992,1993,1994,1995, 1996,1997,2005,2006	9	1996,1997,2006	3
		Moderate	1989	1	1989,1991,1992,1994,2005	5
		Severe	1995,2003	2	1995,2003	2
Barisal	Period 1	Mild	1969,1971,1976,1977,1980,1982	6	1971	1
		Moderate	1985	1	1976,1977,1980,1982	4
		Severe	1972,1975	2	1972,1975,1985	3

	Period 2	Mild	1989,1999,2005,2006	4	1989,1999,2005,2006	4
		Moderate	1994,1996,1997,2000,2003	5	1996	1
		Severe	1992	1	1992,1994,1997,2000,2003	5
Bhola	Period 1	Mild	1970,1975,1982,1985	4	1982	1
		Moderate	1972,1973	2	1970,1973,1985	3
		Severe	1968,1979	2	1968,1972,1979	3
	Period 2	Mild	1989,1995,1997,2000,2006	5	---	0
		Moderate	1994,1996,2003	3	1989,1995,1997,2000	4
		Severe	1992	1	1992,1994,1996,2003	4
Maijdee Court	Period 1	Mild	1974,1977,1979,1980,1985	5	1980,1985	2
		Moderate	1972,1976	2	1977,1979	2
		Severe	1978	1	1972,1976,1978	4
	Period 2	Mild	1994,1996,1997,2000,2005	5	1994,1996	2
		Moderate	1989,2006	2	1997,2000,2005	3
		Severe	1992	1	1989,1992,2006	3
Sylhet	Period 1	Mild	1969,1972,1975,1978,1986	5	1969,1972	2
		Moderate	1980	1	1975,1978,1986	3
		Severe	1971,1973	2	1971,1973,1980	3
	Period 2	Mild	1996,1997,2002	3	---	0
		Moderate	1992,1994,1999,2001,2003,2006	6	1997,2002	2
		Severe	---	0	1992,1994,1999,2001,2003, 2006	6
Srimongal	Period 1	Mild	1974,1979,1982,1987	4	1974,1979,1982	3
		Moderate	1972,1975	2	---	0
		Severe	1971,1985	2	1971,1972,1975,1985	4
	Period 2	Mild	1989,1995,1996,1997,1999,2003	5	1997	1
		Moderate	1994,1998,2006	3	1989,1995,1996,1999	4
		Severe	1992	1	1992,1994,1998,2006	4
Rangpur	Period 1	Mild	1970,1974,1975,1976,1977,1981	6	1970,1977	2
		Moderate	1971	1	1974,1975,1976,1981	4
		Severe	1968,1972	2	1968,1971,1972	3
	Period 2	Mild	1989,1992,1996,1997,2007	5	---	9
		Moderate	2000,2006	2	1989,1992,1996,1997,2007	5
		Severe	1994	1	1994,2000,2006	3
Dinajpur	Period 1	Mild	1974,1975,1978,1980	4	1978,1980	2
		Moderate	1970,1971,1982	3	1974,1975	2
		Severe	1972	1	1970,1971,1972,1982	4
	Period 2	Mild	1989,1997	2	1989	1
		Moderate	1992,2000,2007	3	1997	1
		Severe	1994,2006	2	1992,1994,2000,2006,2007	5

After comparing the SPI prediction with that of RAI, following observations are made:

- Total number of drought years in SPI is found higher in most of the cases than the prediction of RAI. As an example, in Ishurdy, for period 1, SPI detected 11 years of total drought whereas RAI detected 9 years of total drought and for period 2 SPI detected 9 years of total drought whereas RAI detected only 7 years.

- SPI index is more efficient in detecting mild drought than RAI index. For example in Dhaka while the RAI was able to detect 2 years of mild drought the SPI detected 6 years of mild drought in 1968~1987 period and the SPI detected 5 years of mild drought whereas the RAI detected only 3 years.

- RAI shows higher number of severe drought than the SPI prediction. Some of the years which are found in moderate category in SPI prediction, RAI represents them as severe drought. As an example, in Satkhira, the years 1994, 1996 and 1999 (Table 6) experienced moderate drought according to SPI classification for period 2. But according to RAI classification these three years are categorized as severe drought.

Table 7: Comparison of number of total drought years between two periods (period 1:1968~1987 and period 2: 1988~2007)

Station	Number of Total Drought Years			
	SPI		RAI	
	Period 1	Period 2	Period 1	Period 2
Rajshahi	9	10	7	9
Bogra	9	10	7	9
Ishurdy	11	9	9	7
Khulna	8	8	8	7
Satkhira	6	10	5	8
Jessore	7	9	7	8
Dhaka	10	10	9	9
Mymensingh	5	9	5	7
Faridpur	8	8	8	8
Chittagong	8	9	8	9
Rangamati	8	9	8	8
Cox's Bazar	6	9	6	8
Comilla	8	9	6	8
Chandpur	8	9	8	9
Hatiya	9	7	8	7
Sandwip	9	12	8	10
Barisal	9	10	8	10
Bhola	8	8	7	8
Maijdee Court	8	8	7	8
Sylhet	8	9	8	8
Srimongal	8	10	7	9
Rangpur	9	8	9	8
Dinajpur	8	7	8	7

The comparison in number of total drought years between two periods on the basis of SPI and RAI are given in Table 7. After comparing the two periods' drought years (period 1: 1968~1987 and period 2: 1988~2007), following conclusions can be made:

- The numbers of total drought years have increased in period 2 in Rajshahi, Bogra, Satkhira, Jessore, Mymensingh, Chittagong, Comilla, Chandpur, Sandwip, Barisal and Srimongal. The numbers of total drought years have decrased in period 2 in Ishurdy, Hatiya, Rangpur and Dinajpur. In Dhaka and Faridpur the numbers of total drought years have remained unchanged.

- According to SPI analysis the total drought years in Khulna and Maijdee Court have not changed but in RAI analysis they have decreased in Khulna and in increased in Maijdee Court during period 2. The total drought years for rest of the stations have not changed according to RAI classification but they have increased in period 2 in case of SPI classification.

- On the whole, it is found that the total number of drought years have increased from period 1 (1968~1987) to period 2 (1988~2007) for different locations of Bangladesh.

4. CONCLUSIONS

SPI is found to be more efficient in detecting mild drought than RAI. The SPI is to a great extent more suitable for describing meteorological drought in the tropical environment. This is because all previously identified severe drought periods in the study area are all presented by this drought index. RAI shows higher number of severe drought than the SPI prediction. Some of the years which are found in moderate category in SPI prediction, RAI represents them as severe drought.

REFERENCES

Hayes, M. J. (2000), "Revisiting the SPI: clarifying the process". Drought Network News 1: 13-14.

Keyantash, J · and Dracup, J. (2002). "The quantification of drought an evaluation of drought indices". Bulletin of the American Meteorological Society, 83, 1167-1180, 2002 .

Ma'aruf, M., Bamigbade, I. K., Ahmed, A. and Jonathan, I. (2011). "An Evaluation of the Effects of Drought on Groundnut Yields in Kano". The Information Vol. (1& 2) 2011.

Mckee, T. B., Doesken, N.J., and Kleist, J. (1993). "The Relationship of Drought Frequency & Duration to Time Scales". Eighth Conference on Applied Climatology, 17-22 January, Anaheim, California.

Oladipo, E.O. (1985). "A comparative performance analysis of three meteorological drought indices". J. Climatol., 5: 655-664.

Van Rooy M. P., (1965). "A rainfall anomaly index independent of time and space". Notos, 14: 43-48.

Wilhite, D. A. (2000). "Drought as a natural hazard, concepts and definitions". Drought as a global assessment, edited by Wilhite, D. A., l, 3-18, 2000.

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AN ASSESSMENT OF BEST FITTED RAINFALL DISTRIBUTION FOR PROJECTED CLIMATE CHANGE OVER BANGLADESH USING STATISTICAL DOWNSCALING METHOD

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ABSTRACT

Daily rainfall distributions are used extensively for design of engineering works such as municipal drainage systems, highway and railway culverts, and agricultural systems. It also plays an important role in a diverse range of nonstructural problems involving natural hazards associated with extreme events. In this study, attempts have been made to determine best fitted rainfall distribution and future projected rainfall using the best fitted distribution. For this purpose, observed rainfall data are analyzed using 57 different types of distributions considering three types of fitness co-efficient for a better understanding of rainfall extremities. Analysis has been made for the time period starting from 1971 to 2010. A combined ranking order has been applied considering Kolmogorov-Smirnov, Anderson darling and Chi-Squared goodness fit co-efficient for all the selected stations. Then, a statistical downscaling model, SDSM has been used to generate future projected rainfall for each station over Bangladesh. It has been found that, Burr type XII distribution, Birnbaum–Saunders distribution and Log Pearson type IIIdistribution are the best fitted distributions for most of the rainfall stations in Bangladesh. A2 scenario cannot able to capture rainfall distribution of hilly area properly. But it can capture the distribution pattern of relatively flat area like Dhaka city.

Keywords: *daily rainfall, natural hazards, best fit distribution, goodness fit, SDSM.*

1. INTRODUCTION

A good understanding of the pattern and distribution of rainfall is important for water resource management of a country. Knowledge of rainfall characteristics, its temporal and spatial distribution plays a pivotal role in the design and operation of agricultural systems, telecommunications, run-off control, erosion control as well as water quality systems. Urban areas have short time of rainfall concentration, and therefore, the sensitivity of storm water management systems to rainfall input is very quick. Hence, data is mostly needed at a high resolution and daily or hourly rainfall data is most useful. Similarly newer watershed models for urban storm water management utilize the rainfall data at hourly or daily time scale to predict runoff and its effects on storm water storage and conveyance systems. (Dan'azumi, S et al, 2010)

Moreover, Annual rainfall pattern at a location varies from time to time, but it follows a unique best fitted distribution. We know that distribution fitting is the procedure of selecting a statistical distribution that best fits to a data set generated by some random process. In other words, if you have some random data available, and would like to know what particular distribution can be used to describe your data; then distribution fitting is what someone is looking for. The main purpose of using probability distributions is a scientific way of dealing with uncertainty and making informed business decisions. Again, Daily rainfall distribution is significantly important for determining the frequency and trend of rainfall over a particular area.

Dan'azumi, S et al, 2010 said the modeling of rainfall can be classified into three main areas, namely, (1) Stochastic models of rainfall relating to global climate change, (2) Stochastic rainfall models describing the generation of sequence of dry and wet spells, and (3) Models of frequency analysis of rainfall. The effect of global climate change necessitated researches in to the first class of models. They are predictive models that use the existing data to model the trend of future climate as a result of global warming. The second category includes stochastic models of rainfall processes describing the rainfall occurrence and its characteristics. In this category, generation of the sequence of wet and dry spells is mostly conducted with Markov chain and other models

Salami (2004) indicated that Gary and Robert in1971 studied the normal, log-normal, square-root-normal and cube-root-normal frequency distributions of meteorological data for Texas. The results of this research shows that precipitation data conform to the square-root-normaldistribution, while evaporation and temperature data conform to all of the frequency distributions tested. The evaporation, temperature and precipitation data were further fitted to the Gumbel extreme-value and log-Pearson type IIIdistributions. The precipitation data fit the log-Pearson type III (LP3) distribution more adequately than the Gumbel distribution, while both the evaporation and temperature data conform very well to Gumbel distribution.

Lee (2005) studies the rainfall distribution characteristics of Chia-Nan plain area, by using different statistical analyses such as normal distribution, log-normal distribution, extreme value type I distribution, Pearson type III distribution, and log-Pearson type III distribution. Results showed that the log-Pearson type III distribution performed the best in probability distribution, occupying 50% of the total station number, followed by the logNormal distribution and Pearson type III distribution, which accounts for 19% and 18% of the total station numbers respectively.

Olofintoye, O.O et al (2009) reported that the frequency analysisof the largest, or the smallest, of a sequence of hydrologic events has long been an essential part of the design of hydraulic structures. Therefore, the question of better fit among countless probability models used in frequency analysis is always a fresh one. In his study, he made a statistical comparison of currently popular probability models such as Gumbel, log-logistic, Pearson-3, logPearson-3 and log-Normal-3 distributions were applied to the series of annual instantaneous flood peaks and annual peak daily precipitation for 13 flow gauging and 55 precipitation gauging stations in the Seyhan basin, respectively. The parameters of the distributions were estimated bythe methods of moments (MOM) and probability weighted moments (PWM). A detailed Chi-square and Kolmogorov-Smirnov (k-s) goodness -of-fit tests were also applied. According to the evaluations of Chi-squared tests, Gumbel (MOM) for both flow and precipitation stations in the Seyhan river basin were found to be the best models. As a result of the k-s test, log-Normal-3 (MOM) and logPearson-3 (MOM) models were determined to be the best for flow and precipitation stations, respectively.

Fitting a distribution to rainfall data sets provides a compact and smooth representation of the frequency distribution revealed by the available data, and leads to a systematic procedure for extrapolation of frequencies beyond the range of rainfall data set. Rainfall frequency analyses are used extensively for design of engineering works such as municipal drainage systems, highway and railway culverts, and agricultural systems. It also plays an important role in a diverse range of non-structural problems involving natural hazards associated with extreme events. Increasing global temperature due to climate change results change in rainfall patterns all over the globe as they are interacting components of climate. In this study, attempts have been made to determine best fitted rainfall distribution and future projected rainfall using the best fitted distribution. Finally comparison has been made between two climate period to see whether the rainfall distribution is changed with climate or it remains in as previous state.

2. METHODOLOGY

2.1 Data Collection and Analysis

Observed rainfall data are analyzed using 57 different types of distributions considering three types of fitness co-efficient for a better understanding of rainfall patterns. Rainfall data, for 34 locations spread across the Bangladesh (Figure 1), were collected for the time period ranging from 1971 to 2010. The long term daily data was examined and missing records were removed. After quality control and homogeneity test, 32 out of 34 rainfall station data has been selected for the study. Analysis has been made for forty years time period starting from 1971 to 2010. Data has been analyzed into two twenty year time slice and all 57 distributions are fitted considering each of these time chunks to see the goodness of fit. A combined ranking order has been applied considering Kolmogorov-Smirnov, Anderson darling and Chi-Squared goodness fit co-efficient for all the selected stations, (Hanson and Vogel, 2008). These statistical parameters are well representation of goodness of fit of the data sets. After that, a statistical downscaling model, SDSM has been used to generate future projected rainfall for each station over Bangladesh. SDSM (Statistical DownScaling Model) is a decision support tool for assessing local climate change impacts using a robust statistical downscaling technique. Data of A2 (HadCM3 model) scenario for the 30 year period has been calculated with suitable correlation co-efficient. Then, all the result are also fitted and ranked with 57 statistical distributions to check the performance of the model with observe data sets.

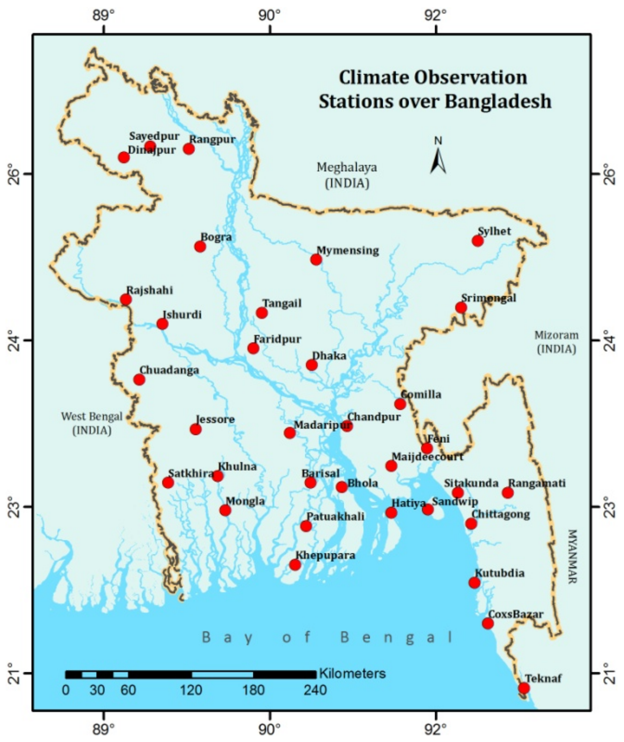

Figure 1: Location of climate observation stations over Bangladesh.

2.2 Goodness of fit test

Three goodness-of-fit tests were performed at 1% significant levels. Note also that X denotes the random variable, and; n, the sample size. The tests are as follow:

2.2.1 Kolmogorov-Smirnov (K-S) Test

This test is used to decide if a sample comes from a hypothesized continuous PDF. It is based on the largest vertical difference between the theoretical and empirical CDF. For a random variable X and sample (x_1, x_2 . . . x_n) the empirical CDF of X ($F_x(x)$) is given by:

$$F_x(x) = 1/n \sum_{i=1}^{n} I(x_i \underline{\propto} x) \tag{1}$$

Where, I (condition) = 1 if true and 0 otherwise. Given two cumulative probability functions F_x and F_y, the Kolmogorov-Smirnov test statistics (D_+ and D_-) are given by:

$$D_+ = max\left(F_x(x) - F_y(x)\right) \tag{2}$$

$$D_- = max(F_y(x) - F_x(x)) \tag{3}$$

2.2.2 Anderson-Darling (A-D)Test

The A-D test compares the fit of an observed CDF to an expected CDF. It gives more weight to the tail of the distribution and the test statistic (A^2) is given by:

$$A^2 = -n - 1/n \sum_{i=1}^{n} (2i-1)[lnF(x_i) + ln\,(1 - F(x_{n-i+1}))] \tag{4}$$

2.2.3 Chi-squared (C-S) Test

This test simply compares how well the theoretical distribution fits the empirical distribution PDF. The C-S test statistic is given by:

$$\chi^2 = \sum_{i=1}^{n} \frac{(O_i - E_i)}{E_i} \tag{5}$$

Where O_i and E_i are the observed and expected frequency for bin i respectively and n is the number of classes. Ei is given by

$$E_i = F(x_2) - F(x_1) \tag{6}$$

and x_1 and x_2 are the lower and upper limits for bin i.

3. RESULTS

Result for goodness-of-fit using K-S, A-D and C-S tests is presented in Table 1 and Figure 2 shows the number of best fitted distribution all over Bangladesh. From table 1 shows that which distribution is best fit for overreach of the stations and also this table helps to understand how rainfall pattern has changed along two climate periods.

Around 45% stations show that best fit distribution model remain same for respective climate period. Birnbaum–Saunders is the most common distribution for the most of stations like Comila, Dinajpur, Faridpur, Tangail, Khulna, Rangpur, Rangamati,Mymensingh, Teknaf, Bhola, Chadpur,Kutubdia and Majidcourt. Some stations have followed other distributions which is also same for the two selected climate . A Satkhira station has maintained lognormal distribution and Sylhet has followed Burr type distribution.

On the other hand, other stations have changed in respect to best fit distribution model. Here, it is revealed that Barisal, cox'bazar , Bogra and Sitakunda had maintained Birnbaum–Saunders distribution for first twenty years but this distribution converted to Log Pearson type III and Log normal in 1991 to 2010.
Log normal distribution was best fit for some stations like Chittagong, Rajshahi and Dhaka over the period 1971 to 1990 but this distribution model has changed for next 20 year period and changed distribution model is Birnbaum–Saunders and Pareto 2. Another distribution model had also occurred which named is Burr type that happened in Hatiya and Jessore stations for first twenty climate years and also revealed in Sandwip in last twenty climate years. Burr type distribution had changed into Birnbaum–Saunders and Log normal.

Table 1: Summary of the best fit statistical distribution at each station for observed data sets

Station	1971 to 1990					1991 to 2010			
	Rank	Best fit distribution model	Kolmogorov -Smirnov	Anderson darling	Chi-Squared	Best fit distribution model	Kolmogorov -Smirnov	Anderson darling	Chi-Squared
Barisal	1	BS	6	1	1	LP3	2	4	1
	2	LNR	5	4	4	LNR	3	3	3
	3	LP3	4	5	6	BS	4	1	4
	4	BXII	2	3	19	BXII	1	5	7
Bhola	1	BS	6	1	1	BS	5	1	1
	2	LNR	3	3	4	LNR	4	4	5
	3	BXII	2	4	8	LP3	3	5	6
	4	PR2	8	2	6	PER6	9	7	7
Bogra	1	BS	5	1	1	LNR	2	3	2
	2	LNR	2	3	4	BS	6	1	1
	3	BXII	4	2	3	LP3	3	5	3
	4	PR2	1	4	5	BXII	1	4	8
Chandpur	1	BS	1	1	1	BS	4	1	2
	2	LNR	3	2	6	LP3	6	5	3
	3	LP3	4	3	5	LNR	2	3	10
	4	PER6	6	7	4	BXII	7	4	4
Chittagong (Patenga)	1	LNR	2	2	2	BS	3	1	1
	2	BS	4	1	1	LP3	5	2	4
	3	LP3	3	4	4	PER6	7	7	2
	4	BXII	1	5	6	BXII	1	5	16
Chuadanga	1	LP3	8	2	1	BS	6	1	1
	2	BS	9	7	2	LNR	3	3	3
	3	BUR	6	6	9	BUR	2	4	7
	4	LNR	5	3	13	PR2	7	2	4

Station	1971 to 1990					1991 to 2010			
	Rank	Best fit distribution model	Kolmogorov -Smirnov	Anderson darling	Chi-Squared	Best fit distribution model	Kolmogorov -Smirnov	Anderson darling	Chi-Squared
Comilla	1	BS	2	1	1	BS	2	1	1
	2	BUR	3	5	8	LP3	3	4	2
	3	LP3	4	2	11	LNR	4	3	3
	4	LNR	5	3	10	BUR	1	5	9
Cox's Bazar	1	BS	7	1	1	LP3	2	4	1
	2	BUR	6	4	3	LNR	3	3	3
	3	PR2	5	5	5	BS	4	1	4
	4	LP3	2	2	13	BUR	1	5	7
Dhaka	1	LNR	3	3	2	PR2	4	2	2
	2	BUR	2	4	3	LNR	3	3	4
	3	PR2	4	2	6	BS	8	1	1
	4	LP3	6	5	4	LP3	5	5	3
Dinajpur	1	BS	4	1	2	BS	6	1	1
	2	LP3	5	6	1	LNR	5	3	2
	3	PR2	2	2	9	LP3	4	5	5
	4	LNR	6	5	3	BUR	3	4	12
Faridpur	1	BS	4	1	1	BS	4	1	1
	2	LNR	2	3	4	LNR	2	3	4
	3	LP3	3	4	5	LP3	3	4	5
	4	PER6	5	7	3	PER6	5	7	3
Feni	1	LP3	4	2	2	BS	3	1	2
	2	PR2	3	3	4	BUR	8	4	3
	3	LNR	2	5	7	LP3	4	2	13
	4	BUR	5	4	5	PR2	9	5	5
Hatiya	1	BUR	4	2	1	BS	3	1	2
	2	LP3	1	3	5	LP3	5	3	8
	3	BS	3	1	8	BUR	6	2	10
	4	PR2	5	4	3	PER6	9	6	3
Ishurdi	1	BS	6	1	1	BS	5	1	2
	2	LNR	1	2	7	BUR	2	5	3
	3	LP3	3	3	11	LNR	1	3	7
	4	PER6	4	6	10	LP3	4	4	5
Jessore	1	BUR	1	5	3	LNR	2	1	1
	2	LP3	5	4	1	BUR	3	5	4
	3	LNR	2	1	8	LP3	4	4	5
	4	BS	6	9	2	BS	5	7	2
Khepupara	1	BS	4	1	6	BS	3	1	1
	2	LNR	2	2	11	LNR	4	3	4
	3	PR2	7	4	7	LP3	5	4	5
	4	BUR	5	5	9	PR2	6	2	9
Khulna	1	BS	7	1	1	BS	5	1	2
	2	LNR	2	4	5	BUR	2	5	3
	3	LP3	4	5	2	LNR	3	3	4
	4	BUR	3	3	7	PR2	7	2	6
Kutubdia	1	BS	6	1	1	BS	1	1	7
	2	LNR	5	4	4	LP3	2	2	9
	3	LP3	4	5	6	FL3P	5	8	1
	4	PER6	9	8	7	PR2	7	4	4
Madaripur	1	PR2	5	2	1	BS	3	1	1
	2	LP3	1	3	6	PR2	6	3	5
	3	LNR	2	6	5	BUR	4	2	11

Station	1971 to 1990					1991 to 2010			
	Rank	Best fit distribution model	Kolmogorov -Smirnov	Anderson darling	Chi-Squared	Best fit distribution model	Kolmogorov -Smirnov	Anderson darling	Chi-Squared
	4	PER6	8	5	4	PER6	9	6	8
Maijdeecourt	1	BS	2	1	1	BS	5	1	1
	2	LNR	4	2	5	LNR	3	3	4
	3	LP3	5	3	3	LP3	4	4	5
	4	PR2	7	4	7	BUR	2	5	8
Mymensing	1	BS	6	1	1	BS	5	1	1
	2	LP3	3	4	6	LNR	2	3	3
	3	PR2	9	2	3	LP3	3	4	5
	4	BUR	5	5	10	BUR	1	5	11
Patuakhali	1	PER6	3	4	4	BS	7	1	1
	2	BS	8	6	1	LNR	4	3	5
	3	PR2	2	1	13	LP3	5	5	4
	4	DAG	4	7	5	BUR	2	4	14
Rajshahi	1	LNR	3	3	1	BS	1	1	2
	2	BS	5	1	2	BUR	2	5	3
	3	LP3	4	4	5	LNR	4	2	4
	4	BUR	2	5	8	LP3	3	4	5
Rangamati	1	BS	6	1	3	BS	5	1	1
	2	BUR	1	5	9	LNR	3	3	5
	3	PR2	8	2	5	LP3	4	4	3
	4	LP3	2	3	16	BUR	2	5	7
Rangpur	1	BS	3	1	1	BS	5	1	1
	2	PER5	10			LNR	3	4	2
	3	LP3	5	4	4	LP3	4	5	4
	4	PR2	7	2	5	BUR	1	3	11
Sandwip	1	LP3	3	3	3	BUR	2	2	5
	2	BS	5	1	5	BS	1	1	10
	3	BUR	7	2	2	JSB	5	3	6
	4	LNR	2	6	10	PR2	7	5	7
Satkhira	1	LNR	3	5	1	LNR	4	3	2
	2	BS	7	1	2	BS	7	1	1
	3	LP3	5	6	3	BUR	2	5	4
	4	BUR	1	4	12	PR2	8	2	5
Sitakunda	1	BS	1	3	2	LNR	3	2	4
	2	BUR	2	5	12	BS	7	1	1
	3	PR2	11	4	4	LP3	4	3	6
	4	DAG	5	9	7	PR2	6	4	10
Srimongal	1	BS	7	1	1	LNR	3	4	1
	2	LP3	4	3	6	LP3	4	5	2
	3	LNR	5	4	5	BUR	2	3	8
	4	PR2	9	2	4	BS	8	1	4
Sylhet	1	BUR	1	1	10	BUR	2	1	5
	2	JSB	5	2	5	PR2	4	2	6
	3	PER6	2	3	11	BS	7	5	2
	4	WEB	6	8	3	PER6	6	3	8
Tangail	1	BS	5	1	4	BS	6	1	1
	2	PR2	3	3	10	PR2	3	2	4
	3	LNR	8	4	6	LNR	2	4	5
	4	LP3	6	2	11	BUR	1	3	8
Teknaf	1	BS	2	1	10	BS	6	1	1
	2	LP3	5	2	7	PR2	3	2	4

Station	1971 to 1990					1991 to 2010			
	Rank	Best fit distribution model	Kolmogorov -Smirnov	Anderson darling	Chi-Squared	Best fit distribution model	Kolmogorov -Smirnov	Anderson darling	Chi-Squared
	3	LNR	4	3	8	LNR	2	4	5
	4	BUR	9	4	5	BUR	1	3	8

*** List of Distribution:** Birnbaum–Saunders = BS; Burr = BUR; Burr type XII = BXII; Dagum = DAG; Johnson SB = JSB, Log Pearson type III = LP3; Lognormal = LNR; Pareto 2 = PR2; Pearson 6 = PER6; Pearson 5 (3P) = PER5; Weibull = WEB.

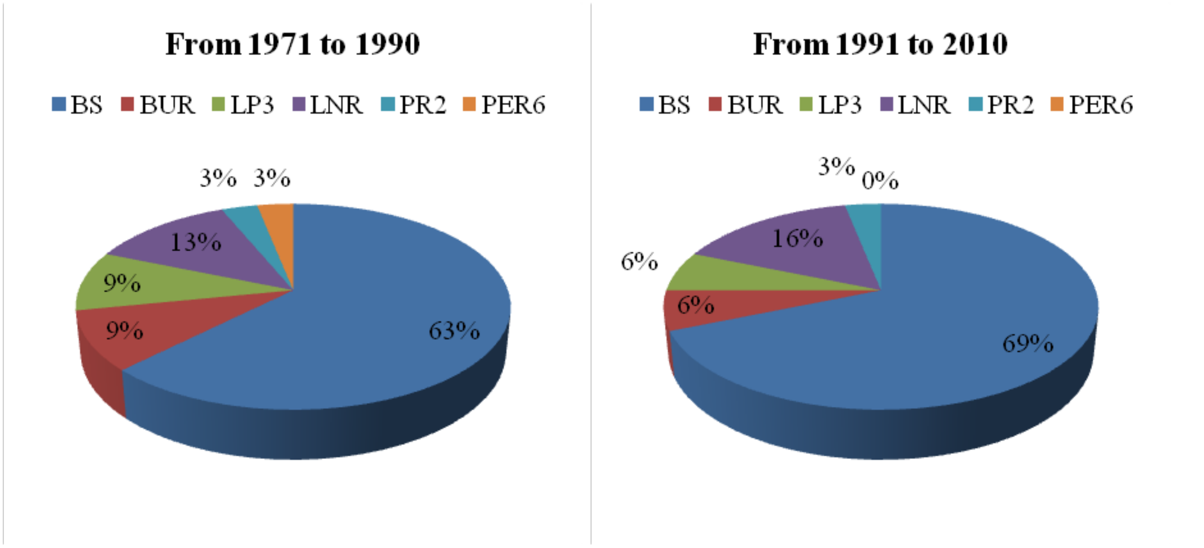

Figure 2: Percentage of best fitted models over Bangladesh for two 20 year climate periods

However, it is observed from figure 2 that, Birnbaum–Saunders distribution is the best fit for most of the stations of the country. The ratio of Birnbaum–Saunders distribution is increased about 6 % for the next climate periods. Again, as 13% stations of Bangaldesh shows lognormal distribution as a best distribution, this percentage is also changeing about one third in recent years. Some distribution model has decreased like Burr type and Log pearson type III which is falled in 3%. On the other hand, Pareto 2 distribution has remained unchanged from 1971 to 1990 and 1991 to 2010. But one distribution model has vanished in next climate period which named is Pearson 6.

Table 2 represents the best fitted model for climate simulation for A2 scenario. From the good ness of fit test, it has been seen that for statistical downscaling shows a different distribution result from observe data sets. For model data sets it has seen that Dhaka station shows similar distribution like observed data. There are many new distribution model which is not matched observed data. The name of new best fit distribution model for climate data which is analysised for SDSM are Gen. Logistic, Log Logistic, Wakeby, Weibull and Gen. Extreme Value.

Instead of different distribution model from observed data, Burr type, Lognormal, Pareto 2, Pearson 6 and Dagum are the most common best fit distribution model for both climate data and observed data.

Table 2: Summary of the best fit statistical distribution at major stations (A2, HadCM3)

Stations	Rank	Best fit distribution model	Kolmogorov-Smirnov	Anderson darling	Chi-Squared
Sylhet	1	GL	1	2	1
	2	LL	7	7	2
	3	WAK	3	1	15
	4	BUR	4	3	16
Comilla	1	BUR	4	4	1
	2	WAK	1	1	10
	3	PAR6	5	6	2
	4	DAG	3	2	14
Rajshahi	1	BUR	4	2	3
	2	WAK	1	1	12
	3	DAG	7	5	6
	4	PAR6	8	6	4
Chittagong	1	WAK	1	1	3
	2	DAG	3	2	8
	3	GL	5	3	5
	4	WEB	2	10	4
Dhaka	1	LR	4	3	7
	2	LP3	9	2	4
	3	PAR2	8	1	6
	4	LL	5	4	10
Khunla	1	GL	3	2	2
	2	WAK	2	1	4
	3	GEV	4	3	3
	4	WEB	1	13	1

* Gen. Logistic=GL, Log Logistic=LL, Wakeby=WAK, Pearson 6=PAR6,Burr= BUR, Dagum=DAG, Weibull=WEB, Lognormal=LR, Pareto 2=PAR2, Gen. Extreme Value=GEV.

4. CONCLUSION

In this study, statistical analysis of daily rainfall was conducted and 57 distributions were employed as candidates for modelling daily rainfall distribution over Bangladesh. Three type of goodness fit parameter has been evaluated throughout the study to access the performance of observed and downscaled model data. Form observed data; it is found that, most of the rainfall stations follow Birnbaum–Saunders and Log-parson 3 distribution rather than conventional gamma distribution. And in some places of Bangladesh it also found that, rainfall distribution pattern is changing along with changing the climate where it is accessed in two twenty year's climate period. Hence, for future decision making in designing hydrological structures, it is necessary to consider appropriate rainfall distribution to calculate return period of a particular rainfall evens. Moreover, Model simulation over major stations revelled that A2 scenario cannot able to capture rainfall distribution of hilly area properly. But it can capture the distribution pattern of relatively flat area like Dhaka city. Future study can be done to find proper bias correction method to adjust these distributions change relating to climate. Also a robust statistical correction is also required for A2 scenario in the hilly area of the country.

REFERENCES

Olofintoye, O.O., Sule, B.F. and Salami, A.W. (2009). "Best–fit Probability Distribution Model for Peak Daily Rainfall of Selected Cities in Nigeria", New York Science Journal, Vol. 2(3), pp. 1-12.

Hanson, L.S. and Vogel, R. (2008). "The Probability Distribution of Daily Rainfall in the United States", World Environmental and Water Resources Congress, http://dx.doi.org/10.1061/40976(316)585.

Papalexiou,S.M., Koutsoyiannis, D. and Makropoulos, C. (2012). "How extreme is extreme? An assessment of daily rainfall distribution tails", Journal Hydrology and Earth System Sciences, Vol. 9, pp. 5757–5778.

Dan'azumi, S., Shamsudin, S. and Rahman, A. A. (2010). "Probability Distribution of Rainfall Depth at Hourly Time-Scale", World Academy of Science, Engineering and Technology, 48.

Balakrishnan, N., Leiva, V. and López, J (2007). "Acceptance Sampling Plans from Truncated Life Tests Based on the GeneralizedBirnbaum–Saunders Distribution", Communications in Statistics—Simulation and Computation, Vol. 36, pp. 643–656.

Lee, C (2005). "Application of rainfall frequency analysis on studying rainfall distribution characteristics of Chia-Nan plain area in Southern Taiwan", Journal of Crop, Environment & Bioinformatics (2). 31-38. 189 Chung-Cheng Rd, Wufeng.

Rahman, M.M., Sarkar, S., Najafi, M.R. and Rai, R.K. (2013). "Regional Extreme Rainfall Mapping for Bangladesh Using L-Moment Technique", Journal of Hydrologic Engineering, Vol. 18(5), pp. 603-615.

Salami, A.W (2004). "Prediction of the annual flow regime along Asa River usingprobability distribution models", AMSE periodical, Lyon, France. Modelling C- 2004, Vol. 65 (2), pp. 41-56

Terink, W., Hurkmans, R.T.W.L., Torfs, P.J.J.F. and Uijlenhoet, R. (2010). "Evaluation of a bias correction method applied to downscaled precipitation and temperature reanalysis data for the Rhine basin", Journal Hydrology and Earth System Sciences, Vol. 14, pp. 687-703.

FRESHWATER CORRIDOR IN SOUTHWEST REGION OF BANGLADESH AND IMPACT OF SLR

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ABSTRACT

Bangladesh has a coastline of over 700 km of which the major part faces the Gangetic delta (also known as southwest region) in Bangladesh. The river Ganga after entering Bangladesh flows by the northern border of this delta and joins the Jamuna (Brahmaputra-Jamuna) just upstream of Aricha. The combined flow known as Padma throws a right bank distributaries – Arial Khan and then flows southeast and meets upper Meghna near Chandpur. The combined flow of the Padma and the upper Meghna is called lower Meghna. In this river reach the lower Meghna throws out a number of right bank distributaries such as Palong, Jayanti, Noya Bhangani, Dharmaganj R. and Tentulia etc. The flows passing through these distributaries feed the rivers Bishkhali, Buriswar and Lohalia-Galachipa and discharges into the Bay of Bengal. River flow measurement data in this area is rare as the rivers are tidal. But if one looks to the shape of surface water salinity contours in the Gangetic delta it becomes clear that Gorai-Modhumati-Baleswar-Haringhata system is Border River that separates highly saline zone to the west (Bagerhat, Khulna and Satkhira districts) and much sweet water zone to the east (Barisal Division). This freshwater zone to the east is due to the Padma freshwater spill feeding the rivers Bishkhali, Buriswar and Lohalia-Galachipa and is called freshwater corridor. There are many facets of Global Climatic change. Most obvious changes that are easily identified are rising trend of temperature and sea level rise (SLR). East coast of Bangladesh from the point of outfall of Feni River in Sandwip estuary runs almost straight to south at the tip of Teknaf (265 km) is not much broken except for the outlets of Karnafuli, Sangu, Matamuhuri and Bogkhali. But the west coast is intersected by large estuaries from the west to east such as Raimongol, Malancha, Sibsa-Passur, Haringhata, Bishkhali, Buriswar and Lohalia. Tentulia, Shahbazpur, and Hatia are considered parts of Meghna estuary. These estuaries are very large and some consider them as arms of sea entering inland. Tides and sea salinity propagates upstream depending on upland flow. This paper is intended to review the impact of sea level rise (SLR) on the salinity pattern in the present day freshwater corridor of Barisal Division.

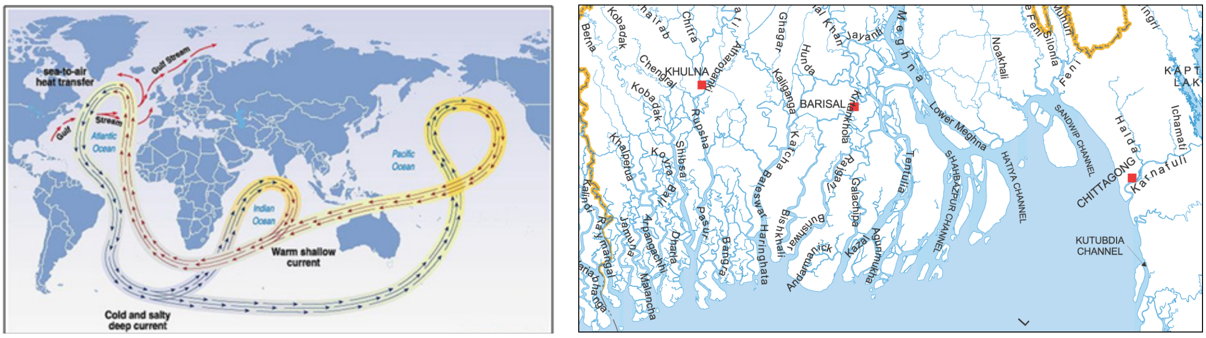
Keywords: *Freshwater corridor, delta, sea level rise (SLR), salinity pattern, global climatic change.*

1. INTRODUCTION

The geographical terrain and almost level topography with dense population have made Bangladesh susceptible to the impacts of Sea Level Rise (SLR). The population is already severely affected by storm surges. Catastrophic events in the past have caused damage up to 100 km inland. A rise in sea level of 1 meter will inundate an estimated 18% of the total land in Bangladesh, directly threatening about 11 % of the population (World Bank 2010). Moreover, the indirect effects of climate change, such as changes in river flows and drainage and the nature of extreme events, could have a large impact on the population, with disproportionate impacts on the rural poor. SLR may also alter the salinity in groundwater and surface water, with corresponding impacts on soil salinity. Saltwater intrusion in groundwater means the gradual or sudden change from freshwater conditions in the ground to saline conditions. Saltwater intrusion can adversely impact the quality and potability of groundwater pumped from wells and the suitability of such water for irrigation. Saltwater intrusion can also cause soil salinization, which may adversely impact crop yields. Saltwater intrusion may occur from saline waters that naturally move up rivers under tidal or storm surge pressures, or from surface flooding associated with storm surges, or from natural processes such as long-term rise in sea level, driving saltwater already underground farther inland. The adverse effects of salinity intrusion in the coastal area especially in the South West region of Bangladesh would be significant on many sectors like agriculture, land fertility, availability of fresh water, existence of the Sundarban forest (the world's largest Mangrove forest), sedimentation rates in the tidal rivers etc. The study focused on the impact of Sea Level Rise (SLR) on the salinity of SW region of Bangladesh. The global conveyor belt thermohaline circulation is driven primarily by the formation and sinking of deep water (from around 1500m to the Antarctic bottom water overlying the bottom of the ocean) in the Norwegian Sea (Figure 1a). This circulation is thought to be responsible for the large flow of upper ocean water from the tropical Pacific to the Indian Ocean through the Indonesian Archipelogo. The two counteracting forcings operating in the North Atlantic control the conveyor belt circulation: (1) the thermal forcing (high-latitude cooling and the low-latitude heating) which drives a polar southward flow; and (2) haline forcing (net high-latitude freshwater gain and low-latitude evaporation) which moves in the opposite direction. In today's Atlantic the thermal forcing dominates, hence, the flow of upper current from south to north.

Bangladesh has a coastline of over 700 km (Figure 1b) of which a major part faces the Gangetic delta (also known as southwest region) in Bangladesh. This coast is intersected by at least seven major estuaries considered as arms of sea entering inland. East coast of Bangladesh from the point of outfall of Feni River in Sandwip estuary runs almost straight to south at the tip of Teknaf (265 km) is mostly unbroken coastline except for the outlets of Karnafuli, Sangu, Matamuhuri and Bogkhali to the Bay of Bengal. The Meghna Estuary (Shahbazpur Channel and Hatia Channel) from east coast of Bhola to the west coast of Sandwip is 70 km wide. The coast of Gangetic delta is thus over 365 km and is intercepted by large estuaries (Figure 1b and Table 1).

The river Ganga after entering Bangladesh flows by the northern border of this delta and joins the Jamuna (Brahmaputra-Jamuna) just upstream of Aricha. The combined flow known as Padma throws a right bank distributaries – Arial Khan and then flows southeast and meets upper Meghna near Chandpur. The combined flow of the Padma and the upper Meghna is called lower Meghna. In this river reach, the lower Meghna throws out a number of right bank distributaries such as Palong, Jayanti, Noya Bhangani, Dharmaganj R. and Tentulia etc. The flows passing through these distributaries also takes the flow of Arial Khan coming from the north and feed the rivers Bishkhali, Buriswar and Lohalia-Galachipa and discharges into the Bay of Bengal.



(a) (b)

Figure 1: (a) Great ocean conveyer belt of salinity (www.grida.no/publications), (b) Bangladesh coast and the costal estuaries.

2. ENVIRONMENT IN BANGLADESH COAST

The districts directly facing sea coasts are Satkhira, Khulna, Bagerhat, Barguna, Patuakhali, Bhola, Lakshmipur, Noakhali, Feni, Chittagong and Cox's Bazar. Because of flat and low topography the districts of Jhalokati and Barisal also experience features of coastal environment.

Estuaries are the large tidal water bodies at the confluence areas of upland rivers with the sea. The river reach in these areas is very wide compared to upland rivers. The size of river section in this river reach is dominated by tidal flow rather than upland discharge. The rivers in this area have properties intermediate of Upland River and the sea. In estuarine areas, seawater is measurably diluted. The upper limit of an estuary is not a fixed point but depends upon upland discharge, tide and the wind force.

2.1 Tide

Tides in Bangladesh coast is semidiurnal i.e. there are two high and two low tides in about 24h 48m. Figure 2 below shows tide level hydrograph for the year 1998. Spring tide and neap tide is clearly visible in the hydrograph. Along with semi-diurnal tide, annual variation of tide level from dry season to monsoon and again the following dry season is also visible. With the high tide saline water from the sea enter into the estuaries. Between the successive estuaries is the low-lying saucer shaped basin lands inundated twice in a day during high tide by the saline water from the sea unless protected by polder dyke and associated infrastructures.

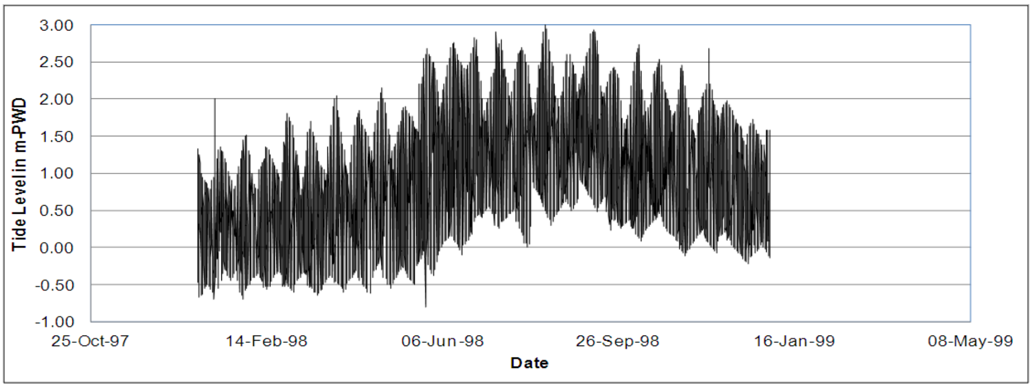


Figure 2: Annual 3 hourly tide level hydrograph of Barishal-Buriswar at SW20 Amtali.

2.2 Salinity

The ratio by weight of total solids to total sample of seawater is approximately what is known as salinity. The ratio by weight of halide ions (Cl -, Br – and I – measured by chemical titration) to total sample of seawater is approximately what is known as chlorinity. Table 1 provides percentage of solids in sea water (Chaw 1968).

Table 1: Components Sea Salinity

Salt Anion		% Total dissolved solids	Salt Cation		% Total dissolved solids
Sodium	(Na+)	30.61%	Chloride	(Cl -)	55.04%
Magnesium	(Mg++)	3.69%	Sulphate	(SO_4 - -)	7.68%
Calcium	(Ca++)	1.16%	Bicarbonate	(HCO_3)	0.41%
Potassium	(K+)	1.10%	Bromide	(Br -)	0.19%
Strontium	(Sr++)	0.04%	Borate	(HBO_3)	0.07%

The relation between salinity and chlorinity can be expressed by Knudsen formula: Salinity=0.03+1.805xChlorinity, where both are expressed in parts per thousands (ppt). Salinity of most seawater is in the range 33 ppt to 37 ppt and an average value of 35 ppt is commonly used.

The estuary areas in Bangladesh may be divided into four types of areas in respect of intensity of salinity:

- Western high saline zone that includes Raimongol, Malancha, Sibsa-Passur and Haringhata estuaries in the districts of Satkhira, Khulna and Bagerhat. These areas are almost totaly cutoff from spill flow from Ganga through Mathabhanga-Bhairab system.
- Central less saline zone that includes Bishkhali, Buriswar and Lohalia flowing through Barisal Division riverine and estuarine area. This area get spill from the Padma and lower Meghna throughout the year.
- Lower Meghna saline estuaries of Tentulia, Shahbazpur and Hatia with year round high freshwater flow of GMB system and also high tide range due to funnel shaped northern end of Bay of Bengal (Matin 2008).
- East coast high saline zones that includes Sandwip, Kutubdia, Moheshkhali and Naf river estuaries. This area is served by freshwater flow from the hill streams and these streams are flashy and the area served is really a narrow strip of delta parallel to folded hills of Chittagong Hill Tracts.

2.3 Coastal embankment project (CEP)

The primary objective of the Coastal Embankment Project was to increase agricultural output by protecting the low-lying coastal lands from regular saline tidal flooding with a view to enhancing the extent of cultivated area as well as its productivity. This was accomplished by empoldering areas with peripheral dykes and closures of and appurtenant structures on tidal creeks.

In 1958, the erstwhile government of East Pakistan started a programme for construction of coastal embankments and polders through its Irrigation Department and the East Pakistan Water and Power Development Authority (EPWAPDA) with the assistance of USAID. Subsequently, after independence in 1971, the Government of Bangladesh (GOB) through the Bangladesh Water Development Board (BWDB) resumed the programme to complete the works. BWDB completed about 4800 km of embankment with some 2500 allied structures, including about 1500 regulators, to protect an area of about 1.4 million ha, of which 1 million ha is cultivable. Altogether 125 polders, including sub-polders, were constructed under the Coastal Embankment Project. The polders have not only delivered benefits for over three decades in terms of higher agricultural returns, but have also brought about overall environmental enhancement due to exclusion of saline tidal flooding, resulting in better living conditions. Communications have also improved. However, from the early-1980s the problem of drainage congestion surfaced quite prominently because of the gradual siltation of channels and out-fall creeks due to a change in tidal prism. This specially occurred in the western part of the delta now considered moribund (e.g. Beel Dakatia). Lack of upland flow (Ganges flow reduction) and the construction of embankments are cited as the major factors for accelerated siltation.

2.4 Coastal embankment project (CEP)

The Cyclonic storms are a major feature of the Bangladesh climate. Records go back more than 400 years, though early ones are patchy. In the 200 years to 1999 the coastal zone was affected by at least 59 such storms, though recent decades have seen higher frequency, with 39 storms since 1948. The few worst ones have each killed more than 100000 people.

A cyclone is a region of unusually low atmospheric pressure. The direct consequence of this, over the Bay of Bengal and the coastal zone of Bangladesh, is a slight rise in sea water level. Further consequences of a large cyclone include:

- winds of up to about 240 km/h rotating around a central eye, up to 60 or 70 km in diameter, which moves along a path (the cyclone's track) that can be only partially forecast;
- a storm surge of raised water levels, with high waves, which is amplified as it moves up the Bay of Bengal and inland, with water levels rising by up to 6 or 7 m in extreme cases;
- exceptionally heavy rainfall, which of course also tends to cause flooding.

Damage and loss of life in a cyclonic storm surge are mainly caused by water depths, water currents, waves and wind. They can be mitigated by appropriate measures taken in advance, like land use zoning, warning systems, embankments, safe havens, and robust structures (avoiding loose corrugated iron roof sheets that are dangerous in high winds, for example). Many of these are discussed in later chapters of this report.

2.5 Effect of SLR in Bangladesh

The geographical terrain and almost level topography with dense population have made Bangladesh coast susceptible to the impacts of Sea Level Rise (SLR). The population is already severely affected by storm surges. Catastrophic events in the past have caused damage up to 100 km inland. The adverse effects of salinity intrusion in the coastal area especially in the South West region of Bangladesh would be significant on many sectors like agriculture, land fertility, availability of fresh water, existence of the Sundarban forest (the world's largest Mangrove forest), sedimentation rates in the tidal rivers etc.

2.5.1 Analysis of recent sea level data of Bangladesh coast

Tide level data for Hiron point, Khepupara and Charchanga near the sea coast were obtained from BIWTA. Datum of Tide level collected by BIWTA is local datum called Chart datum defined on the basis of tide characteristics of the locality. From the basic data three annual time series were developed. These are: (a) Annual Highest Tide, (b) Annual average tide series and (c) Annual lowest tide series. Time trend analysis of annual average tide (Table 2) is presented below:

Table 2: Time trend analysis of annual highest, average and lowest tide

Station Name	Period	Annual Highest Tide mm/Year	Annual Average Tide mm/Year	Annual Lowest Tide mm/Year
Hiron Point	1977 to 2007	5	5	-1.7
Khepupara	1977 to 2009	4.9	7.9	12
Charchanga	1979 to 2009	5.4	12.1	18.8

2.6 Climate change scenarios

The climate change scenarios and potential effects of climate change on Bangladesh during period up to 2050 are presented in Table 3. Most development projects of Bangladesh have a planning horizon of 30 years or less, while a few have a planning horizon of 50 years or slightly more. The climate change impact studies in this communication have been mainly based on the climate change scenarios developed for 2030 and 2050 as reported by a study carried out by the World Bank in 2000.

According to the above scenarios, the magnitude of these changes in climate may appear to be very small. But, if added to existing climatic events (such as floods, droughts, and cyclones), these could substantially increase the magnitude of these events and decrease their return period. For example, a 10 percent increase in precipitation may increase runoff depth by one-fifth and the probability of extreme wet year by 700 percent. Thus, within the planning horizon for development activities, it is quite possible that there could be a significant increase in the intensity and frequency of extreme climate events in Bangladesh (World Bank 2000).

Table 3: Climate change scenarios for Bangladesh by 2030 and 2050

Year	Sea Level Rise (cm)	Temperature Increase (°C)	Precipitation Compared to 1990 (%)	Changes in Evaporation
2030	30	+ 0.7 in monsoon +1.3 in winter	-3 in winter +11 in monsoon	+0.9 in winter +15.8 in monsoon
2050	50	+1.1 in monsoon +1.3 in winter	-37 in winter +28 in monsoon	0 in winter +16.7 in monsoon

Source: World Bank 2000.

3. FRESHWATER CORRIDOR

Bangladesh coast of Gangetic delta is traversed by large estuaries and the terrains between estuaries are mostly below high tide. As the tide propagates up land through these estuaries the rivers and the terrain is flooded by saline water from the sea twice in a day unless protected by Polder dikes and appurtenant structures. But in Barisal Division the rivers Bishkhali, Buriswar and Lohalia/Galachipa get spill flow from lower Meghna right bank distributaries and obstructs propagation of salinity upstream and hence the river system in Barisal Division may be called Freshwater Corridor. This is evident from the salinity (annual maximum EC for the year 2002 a typical year) contour Figure 3.

3.1 Salinity contours

Figure 3 shows salinity (EC values) contours of annual highest salinity for the year 2002. In the Figure 3, black colored arrow shows the way salinity from the Western high saline zone seems to propagate in a north-easterly direction, the gray colored arrow shows propagation of salinity in the Lower Meghna estuary and the white colored arrow shows obstruction impact of high freshwater flow in the freshwater corridor against the propagation of salinity in the upstream.

The flows passing through the right bank distributaries of Lower Meghna feed the rivers Bishkhali, Buriswar and Lohalia-Galachipa and discharges into the Bay of Bengal. It is this freshwater spill flow of the Meghna through these distributaries that deters propagation of salinity into the upstream. If one looks to the shape of surface water salinity contours in the Gangetic delta it becomes clear that Gorai-Modhumati-Baleswar-Haringhata system is the border river that separates high saline zone to the west (Bagerhat, Khulna and Satkhira districts) and much sweet water zone to the east (Barisal Division). This freshwater zone to the east is due to the Padma-Lower Meghna freshwater spill (Figure 4) feeding the rivers Bishkhali, Buriswar and Lohalia-Galachipa and is called freshwater corridor. Figure 5 shows salinity (EC values) contours of annual highest salinity from the year 2002 to 2008.

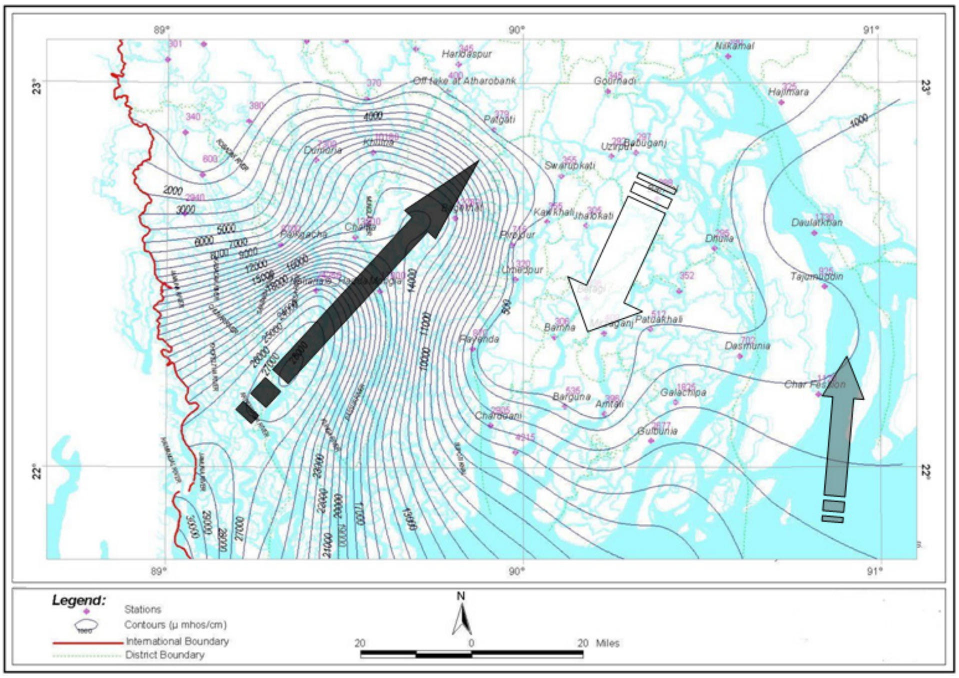


Figure 3: Typical salinity (EC values) contours, 2002.

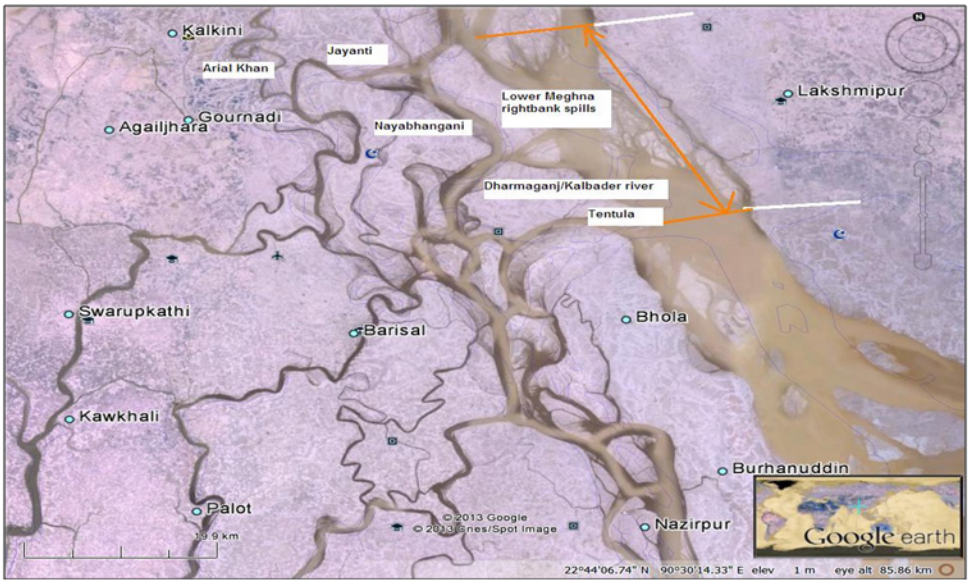


Figure 4: Spill from Lower Meghna feeding freshwater corridor in Barisal Division.

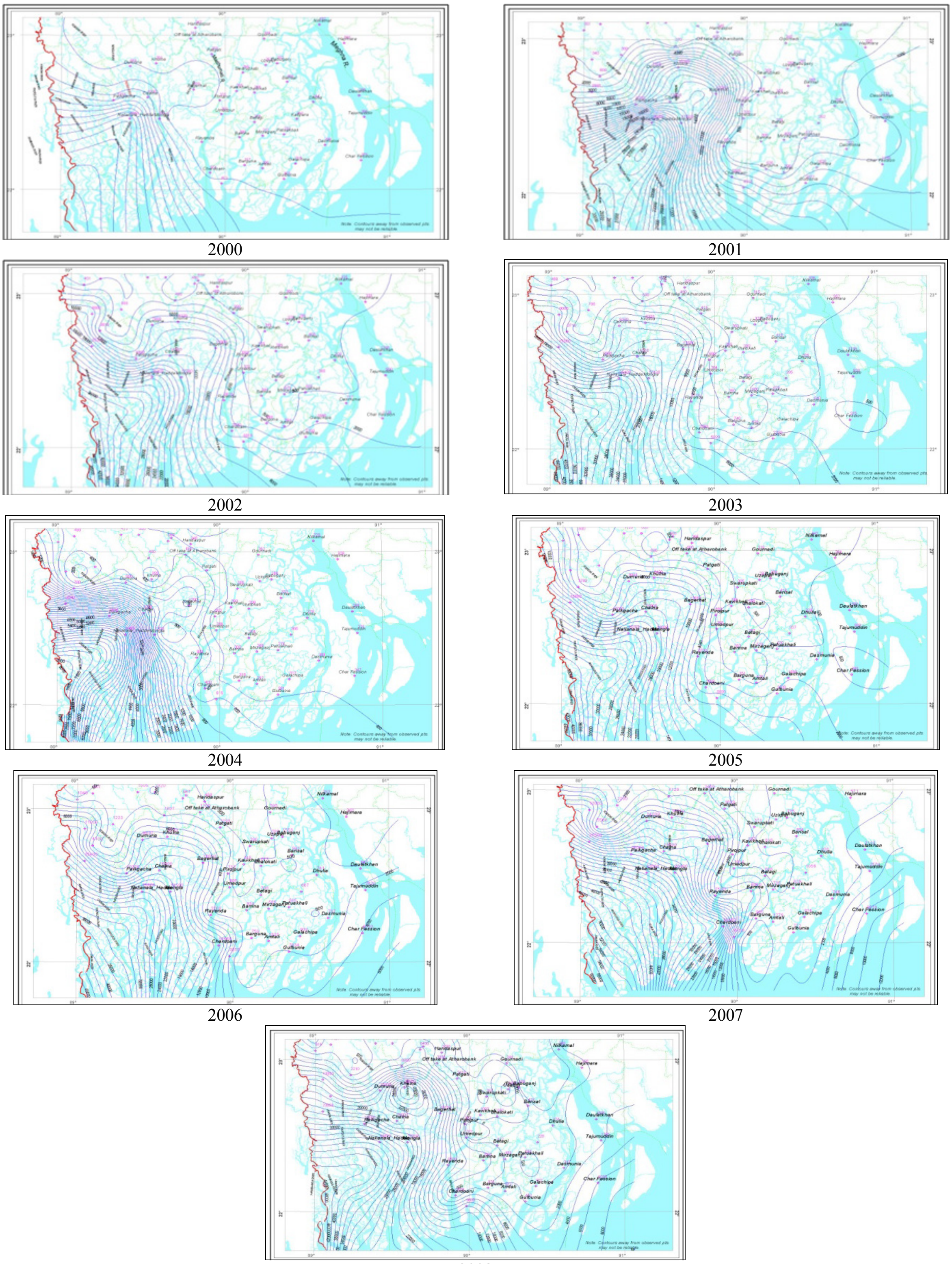


Figure 5: Salinity (EC values) contours, from 2000 to 2008.

Table 4 below shows the list of major rivers in the Gangetic delta, salinity (EC) monitoring stations in the rivers and location of the stations in the districts and upazilla, distance of the stations from the sea face, duration in days of salinity above 2250 µmhos/cm and highest value of EC. In the districts of Satkhira, Khulna and Bagerhat although the monitoring stations are located 80 to over 150 km from sea face - duration of salinity above critical value is 74 to 296 days and maximum salinity ranges from 3,610 to 25,200 µmhos/cm whereas in the Baleswar - the boundary river at locations 21 to 84 km the said duration is 0 to 74 days with maximum salinity value of 3610 µmhos/cm. Figure 6 represents the EC values with distance from sea face in 2002 according the Table 4.

Further to the east in the rivers Bishkhali, Buriswar and Lohalia-Galachipa out of 10 salinity monitoring points only at 2 points salinity duration above 2250 µmhos/cm was 74 days. These two points are only 15.99 km and 42.77 km from sea face. Maximum salinity at these two points were 5010 and 2700 µmhos/cm. For the remaining 8 points – distance from sea face ranging from 34.28 km to 130.75 had maximum salinity from 344 to 1840 µmhos/cm well below critical salinity.

Table 4: Summary of duration of salinity above 2250 µmhos/cm in year 2002

River	District	Upazila	Station	Distance from sea face (km)	Duration days above 2250 µmhos/cm	EC max µmhos/cm
						High Saline Zone
Betna, Arpangasia	Satkhira	Satkhira Sadar	SW24 Benerpota	155.71	74	3610
Sibsa	Khulna	Dacope	SW259 Hadda	87.95	296	25200
Rupsa-Pasur	Khulna	Khulna Sadar	SW241 Khulna	134.49	91	8120
Rupsa-Pasur	Bagerhat	Mongla	SW244 Mongla	89.15	162	17380
						Boundary between High Saline Zone and freshwater corridor
Baleswar	Pirojpur	Pirojpur Sadar	SW107 Pirojpur	84.00	0	680
Baleswar	Bagerhat	Sarankhola	SW107.2 Rayenda	45.00	60	3610
Baleswar	Barguna	Patharghata	SW108 Char doani	21.71	30	2930
						Freshwater Corridor
Bishkhali	Barguna	Bamna	SW38 Bamna	61.39	0	344
Bishkhali	Barguna	Barguna Sadar	SW38.1 Barguna	34.28	0	660
Bishkhali	Baguna	Patharghata	SW39 Patharghata	15.99	74	5010
Barisal-Buriswar	Barisal	Barisal Sadar	SW18 Barisal	130.75	0	405
Barisal-Buriswar	Patuakhali	Mirzaganj	SW19 Mirzaganj	63.63	0	475
Barisal-Buriswar	Barguna	Amtali	SW20 Amtali	38.93	0	384
Lohalia/ Galachipa	Patuakhali	Patuakhali Sadar	SW184 Patuakhali	76.26	0	505
Lohalia/ Galachipa	Patuakhali	Galachipa	SW185 Galachipa	42.77	0	1846
Lohalia/ Galachipa	Patuakhali	Galachipa	SW185.1 Gulbunia	26.94	74	2700
Tentulia	Patuakhali	Dasmunia	SW159 Dasmunia	68.38	0	717

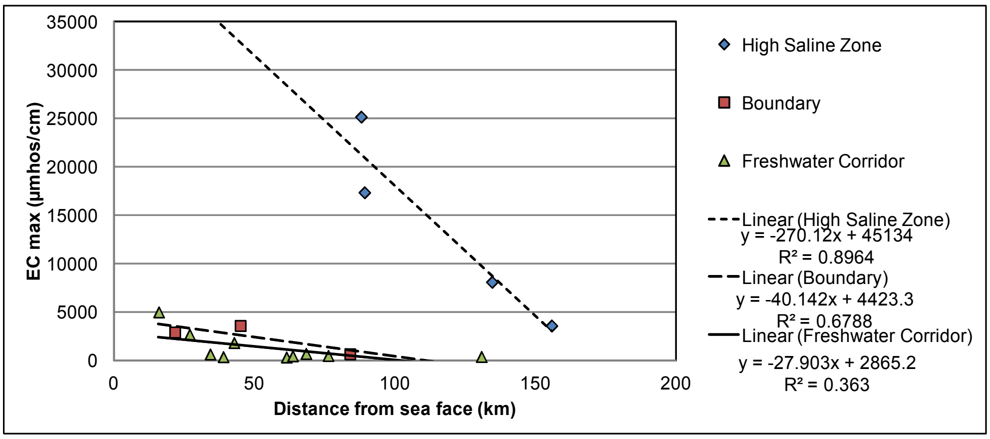
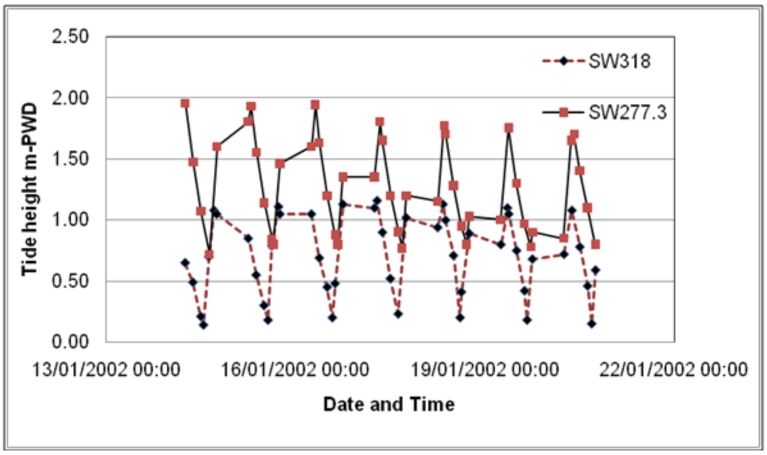


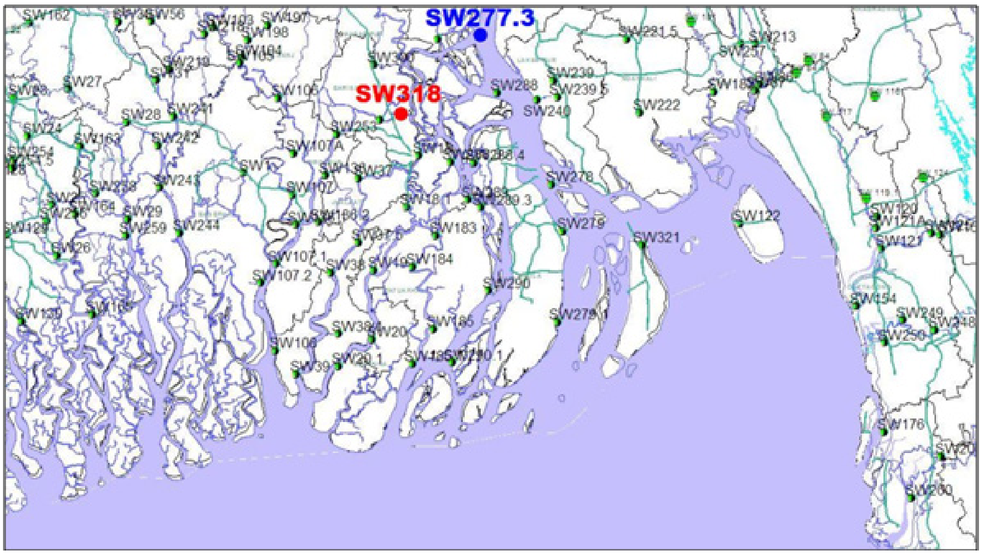
Figure 6: EC values with distance from sea face in 2002.

3.2 Hydraulics of freshwater corridor

In order to understand the flow hydraulics of the Freshwater Corridor, Figure 7(a) represents the day time three hourly tide levels as available at SW277.3 Nilkamal on the Meghna and SW318 Babuganj on the Babuganj spill channel and the locations are shown in Figure 7 (b):



(a)



(b)

Figure 7: Superimposed Tide Hydrograph 14-20 Jan/2002, SW277.3 Nilkamal Lower Meghna and SW318 Babuganj River Babuganj.

Distance between the two stations (Figure 7b) is 61.35 km and slope computed using tide level comes to about 0.00000940 m/m on average with a maximum of 0.00002119 m/m and a minimum of 0.00000228 m/m for the period 14/01/2002 06:00 to 20/01/2002 16:30 randomly selected for a spring tide. This shows that substantial spill flow may be generated even during dry season.

It is known that high tide range occur at the funnel shaped end of Bay of Bengal and with sea level rise this tide range is likely increase further and will continue to feed the freshwater corridor. It may further be noted that while the major river distributaries like Old Brahmaputra, Dhaleswari, Ichamati, Gorai are deteriorating the Lower Meghna right bank distributaries are not showing any deterioration in the recent past. The likely cause is the thrust of tide from the sea side is pushing more flow into the Freshwater Corridor. This is a ray of hope that even with the threat of SLR that Barisal Division will remain sweet with flow from Freshwater Corridor.

4. CONCLUSION

In spite of many benefits of Polders, one major difficulties of sedimentation at the outlet of drainage channels is that the Polders have cut down the tide prism that creates high flow down the channel during ebb flow. Absence of tide prism causes sedimentation at the outfall channel. Moreover the appurtenant hydraulic structures are not to be considered static but have to be operated to create the most congenial environment for agriculture and fisheries while minimizing adverse side effects.

For western high saline zones pilot closure of estuaries may be thought to obstruct incursion of salinity and storm surges and convert estuaries into fresh water lakes. This will also solve the present water logging problem in the western belt of Satkhira by eliminating sediment laden water from the coast.

In this paper, it is tried to establish that, comparatively freshwater environment in Barisal district is due to Freshwater Corridor created by the impact of tides at funnel end of Bay of Bengal on the upland river flow of Lower Meghna.

REFERENCES

Chow, T. V. (1964). Handbook of Applied Hydrology, McGraw-Hill, Inc, 1st edition, New York.

Matin, M. A. (2008). “Risk Assessment and Evaluation of probability of Extreme Hydrological Events: Case Study from Noakhali Sadar and Subarnachar Upazilas”, IUCN Bangladesh Country Office, Dhaka, Bangladesh, viii+76 pp.

World Bank (2000). “Bangladesh: Climate Change & Sustainable Development”, Rural Development Unit, South Asia Region, Report No. 21104, Dhaka, Bangladesh, 19 December.

World Bank (2010). “Implications of Climate Change for Fresh Groundwater Resources in Coastal Aquifers in Bangladesh”, South Asia Region, Sustainable Development Department, Agriculture and Rural Development Unit Report No. 56556, Dhaka, Bangladesh, 1 February.

IMPACT OF MIXED CROPPING ON GROUNDWATER BASED IRRIGATION IN SOUTH-WEST REGION OF BANGLADESH

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ABSTRACT

In Bangladesh, during the dry season of the year (November- April), when rainfall is low, the irrigation is done by extracting groundwater. Over the years, this continuous groundwater pumping causes the depletion of groundwater table. Due to the proximity to the sea, the lowering of groundwater initiates saltwater intrusion into the coastal aquifers and this affects the crop production significantly. Generic models were established to analyze the inter-relationships among different cropping patterns. Crop Irrigation models were established in CROPWAT by using CLIMWAT climate data which is a climatic database developed by Food and Agriculture Organization (FAO) of United Nations. Two meteorological stations in the South-West Bangladesh were selected (i.e. Khulna and Barisal) and two cropping patterns were considered, i.e. rice cultivation throughout the whole year and mixed cultivation of rice and vegetables. Crop Water Requirements (CWR), Irrigation Required (IR) and Groundwater Recharge were calculated and compared for the two different cropping patterns. The analysis showed that irrigation requirement decreases significantly, especially in dry season of the year, if the farmers switch their cultivation practice from single cropping to mixed cropping. As a consequence, the groundwater recharge will increase and it will reduce the extent of saltwater intrusion inland.

Keywords: *Mixed Cropping, Crop Water Requirement, Groundwater, Saltwater Intrusion, Agriculture in Bangladesh*

1. INTRODUCTION

Bangladesh, located on the northern littoral of the Bay of Bengal, has a total coastline length of 710 km (CZPo, 2005). Agriculture is a key economic activity for the 40 million inhabitants in the coastal region (Roy, 2011). The standard of living for this vast population is largely dependent on the growth and sustainability of agricultural production. The south-west of Bangladesh has adequate freshwater during the wet seasons (June to October) due to excessive rainfall, although which is sometimes responsible for river flooding as well (Rasid & Paul, 1987). However during the dry season (November to April) the rainfall is very low and the lack of flow in the rivers make it almost impossible to use surface water for irrigation purposes. To make the best use of the fertile arable lands, the farmers use groundwater to irrigate their crops during the dry season. The elevation of the groundwater table as well as the quality of groundwater is the key issue for groundwater use. The recent trend of depletion of groundwater table is the result of overpumping and decline of overall recharge in that area (Shamsudduha, et al., 2009).

In the future, groundwater availability in the south-west region of Bangladesh will be challenged because of the consequences of climate change. Bangladesh is the third most vulnerable country to climate change in terms of population and among the top ten considering the percentage of people living in the low-lying coastal zones (Pender, 2008). The change in the climate condition may influence the existing agricultural system by altering the rainfall distribution over the year and subsequent change of groundwater recharge. In that case, the current cultivation practice may require modification by changing cropping pattern to cope with the new scenario. Therefore, it is important to find out the most suitable cropping pattern by analyzing different alternatives to cope with the newly arise situation in the future. This study aims to compare the suitability of two different cropping patterns (i.e. rice cultivation throughout the whole year and mixed cultivation of rice and vegetables) in terms of groundwater extraction for Khulna and Barisal region by calculating Irrigation Requirements for different seasons by using CROPWAT models considering the plant characteristics, soil type, seasonal rainfall and evapotranspiration. CROPWAT is a computer program for irrigation planning and management (FAO, 2012). CLIMWAT 2.0 database was used as a source of climate data.

2. METHODOLOGY

2.1 Data Source

Climate data (i.e. rainfall, evapotranspiration, temperature) was extracted from CLIMWAT 2.0 database which is an agro-climatic database developed and published jointly by the Water Resource, Development and Management Service (AGLW) and the Environment and Natural Resources Service (SDRN) of Food and Agriculture Organization (FAO) of United Nations. The database is provided by agro-meteorological group of FAO. This database can be used in combination with the computer program CROPWAT 8.0 to calculate CWR, IR and irrigation scheduling for various range of crops in different climatological stations worldwide. It contains long-term monthly mean values of seven parameters observed over 5000 stations throughout the whole world. The database was made on the basis of available dataset from the period of 1971-2000. For some stations the dataset may be broken due to unavailability, e.g. 1961-70 and 1992-2000, but they contain at least 15 years of data (FAO CLIMWAT, 2012). CLIMWAT 2.0 provides: (i) Mean daily temperature (maximum and minimum) in °C (ii) Mean relative humidity in % (iii) Mean wind speed in km/day (iv) Mean sunshine hours per day (v) Mean solar radiation in MJ/m2/day (vi) Monthly rainfall (total and effective rainfall) in mm/month and (vii) Reference evapotranspiration calculated with the Penman-Monteith method in mm/day.

Different types of soil properties (i.e. moisture, infiltration rate etc.) and crop properties (growing period, crop factor, rooting depth etc.) were available as a database within the CROPWAT 8.0 program and were used to accomplish the model design and construction. Generic soil type, that represents south-west region (mainly Khulna and Barisal region) of Bangladesh, was selected in the model by going through different relevant literatures.

2.2 Model Design and Construction

2.2.1 Exporting Data To CROPWAT From CLIMWAT

Models were created for two climatological stations (Khulna and Barisal) available in CLIMWAT2.0. In spite of having the same overall climate pattern, there are variations in rainfall and evapotranspiration between these two places. To investigate the effects of this dissimilarity, separate models were constructed for Khulna and Barisal and comparison was made. The locations of the two climatological stations is shown in Figure 1 and the longitude, latitude and altitude is presented in Table 1.

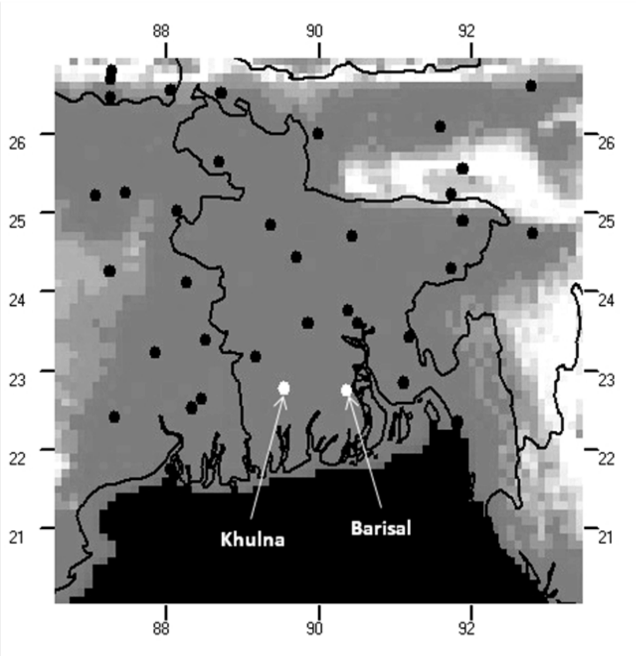

Figure 19: The Location of Two Climatological Stations Barisal and Khulna (shown in CLIMWAT 2.0)

Table 2: Location of Climatological Stations

Station name	Latitude	Longitude	Altitude (above MSL)
Khulna	22.78˚ N	89.53˚ E	4 m
Barisal	22.75˚ N	90.36˚ E	4 m

The models were constructed by importing the available climate data of the above mentioned two stations from CLIMWAT 2.0 to CROPWAT 8.0. The database for each station contains two files. They are the ET_o file (.pem) and rain file (.cli) which corresponds to evapotranspiration and rainfall data respectively. The compariosn of total rainfall and ET_o between Khulna and Barisal stations are shown in Figure 2.

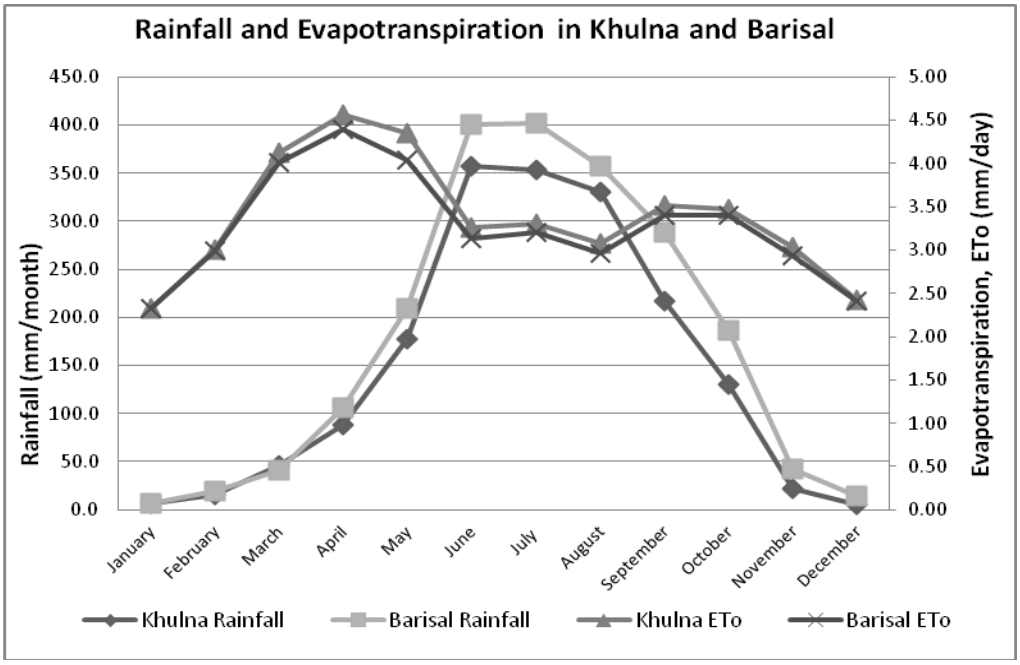

Figure 20: Comparison of Rainfall and ET_o of Between Khulna and Barisal

2.2.2 Selection of Land Area For Each Crop

Farmers do not cultivate crops on the entire land area. It was assumed that uncultivated area is covered with vegetation that is modelled as turf grass. These areas are not to be irrigated. According to Haque (2006), out of 2.85 million hectares of the coastal and offshore areas about 0.83 million hectares are arable lands. This is approximately 30% of the total area. Among this arable land rice is produced in 74% of the cropped area (Islam, 2000). The three cropping system is prevalent in southwest region that includes three major rice types namely, Aman, Boro and Aush that covers 51%, 39% and 10% of the rice cultivation area (Oryza, 2011). So, Aman occupies 51×0.74 = 37% of total arable land. The percent coverage of each type of crop is shown in Figure 3.

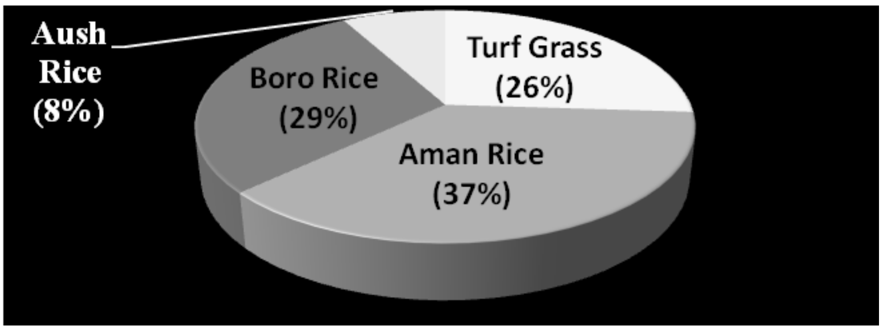

Figure 21: Usual Percent Coverage of Crops in the Arable Land Area

2.2.3 Selection of Soil Type

The majority part of this region has sedimentary soils and the sediments are mainly non-calcareous clays but are silty and slight calcareous on riverbanks (Banglapedia, 2006). This area faces soil salinity problem especially during dry seasons (November to March) when rainfall is very low. Among the classification of the soils available in CROPWAT, "medium soil" was selected to represent the mixture of clay, silt and humus found in the soil of the floodplain. This type of soil ("loam") contains roughly equal concentration of sand, silt and clay (Wikipedia , 2012). A medium soil contains the necessary nutrients for plants, holds sufficient water to make them available to the root and allows good drainage and considered very fertile (Soil, 2012). The soil data that was used is shown in Table 2.

Table 3: Properties of Medium (Loam) Soil [Source: CROPWAT 8.0]

Soil Type	Medium (loam)
Total available soil moisture (TAM)	290.0 mm/meter
Maximum rain infiltration rate	40 mm/day
Maximum rooting depth	900 cm
Initial soil moisture depletion (as % of TAM)	0%
Initial available soil moisture	290 mm/meter

2.2.4 *Selection of Cropping Pattern*

Two cropping models for each climatological station were constructed. In the first model, only rice was planted and harvested three times in a year. In the second model, vegetables and rice both are used and it was assumed that rice is planted and harvested twice in a year with vegetables grown during the dry season during the months of December to March (Table 3).

Table 4: Cropping Pattern for Two Different Models

Model 1 (Rice only)			Model 2 (Rice and vegetables)		
Crop name	Planting date	Harvesting date	Crop name	Planting date	Harvesting date
Rice (Aman)	25/08	22/12	Rice (Aman)	25/08	22/12
Rice (Boro)	25/12	23/04	Vegetables	25/12	29/03
Rice (Aush)	25/04	22/08	Rice (Aush)	25/04	22/08
Turf grass	25/04	24/04	Turf grass	25/04	24/04

2.2.5 *Crop Properties*

The crop properties that will eventually influence the CWR and IR in different stages of plant growth include crop coefficient (K_c), number of days for different growth stages, rooting depth, puddling depth during land preparation, percent of nursery area required during nursery period, critical depletion and yield response factor. The usual duration for rice from transplanting time to the harvesting time is four months and it was set 150 days in the model. For small vegetables this duration is 95 days. Diffrent properties of rice and vegetables are shown in the Appendix.

2.2.6 *Calculation of Crop Water Requirement (CWR)*

CWR is the "amount of water required to compensate the evapotranspiration loss from the cropped field" (Allen, et al., 1998). In other words, it is the amount of water needed by the plants for optimal growth. It always refers to "a crop grown under optimal conditions, i.e. a uniform crop, actively growing, completely shading the ground, free of diseases, and favorable soil conditions (including fertility and water). The crop thus reaches its full production potential under the given environment" (Brouwer & Heibloem, 1986).

The CWR for a particular type of plant can be formulated as,

$$CWR = ET_o \times K_c \tag{1}$$

Here, CWR = ET_{crop} or crop evapotranspiration (mm/day), ET_o = Reference evapotranspiration (mm/day) [evapotranspiration of reference grass crop] and K_c = Crop factor which is dependent on crop type.

The reference evapotranspiration (ET_o) can be calculated using FAO Penman-Monteith method by combining (i) Penman-Monteith equation, (ii) the equations of aerodynamics and (iii) surface resistance (Brouwer & Heibloem, 1986).

$$ET_o = \frac{0.408\Delta(R_n - G) + \gamma \frac{900}{T + 273} u_2(e_s - e_a)}{\Delta + \gamma(1 + 0.3u_2)} \tag{2}$$

Here, ET_o = reference evapotranspiration [mm day^{-1}], R_n = net radiation at the crop surface [MJ m^{-2} day^{-1}], G = soil heat flux density [MJ m^{-2} day^{-1}], T = mean daily air temperature at 2 m height [°C], u_2 = wind speed at 2 m height [m s^{-1}], e_s = saturation vapour pressure [kPa], e_a = actual vapor pressure [kPa], e_s - e_a = saturation vapor pressure deficit [kPa], D = slope vapor pressure curve [kPa °C^{-1}] and γ = psychrometric constant [kPa °C^{-1}].

2.2.7 *Calculating* Irrigation Required For Plants

The Irrigation requirement for crop production can be defined as "the amount of water, in addition to rainfall, that must be applied to meet a crop's evapotranspiration needs without significant reduction in yield" (Smajstrla & Zazueta, 1996). Irrigation required (IR) can be formulated as

$$IR = ET_{crop} - P_e \tag{3}$$

Here, IR = Irrigation required, ET_{crop} = Crop evapotranspiration and P_e = Effective rainfall.

Effective rainfall (P_e) is the difference between total rainfall and the actual evapotranspiration. In irrigation, it is the portion of rainfall that remains in the soil for the utilization of plant for its germination. In reality, there are water losses in field conditions.

$$IR = ET_{crop} + SAT + PERC + WL - P_e \tag{4}$$

Here, SAT is the amount of water needed to saturate the soil for land preparation by puddling. It depends on soil type and root zone depth. PERC stands for percolation and seepage losses which is a function of soil type. WL is the amount of water needed to establish a water layer.

2.2.8 *Calculation of Groundwater Recharge*

Groundwater recharge was calculated using the following formula

$$Recharge = Rainfall - \sum ET_c \times \%\ of\ land\ area \tag{5}$$

From the CLIMWAT 2.0 database, the annual rainfall for Khulna and Barisal was found to be 1748 mm and 2070 mm respectively. The monthly distribution of rainfall is provided in Table 5 in the Appendix. The crop evapotranspiration for different crops are shown in Table 4.

Table 5: Crop Evapotranspiration (ETc) Data from CROPWAT 8.0

<table>
<tr><th colspan="5">Model 1 (Rice only)</th><th colspan="5">Model 2 (Rice and vegetables)</th></tr>
<tr><td rowspan="3">Crop Type</td><td colspan="4">Crop Evapotranspiration, ET_c (mm)</td><td rowspan="3">Crop Type</td><td colspan="4">Crop Evapotranspiration, ET_c (mm)</td></tr>
<tr><td colspan="2">Khulna</td><td colspan="2">Barisal</td><td colspan="2">Khulna</td><td colspan="2">Barisal</td></tr>
<tr><td></td><td>Total</td><td></td><td>Total</td><td></td><td>Total</td><td></td><td>Total</td></tr>
<tr><td>Aman Rice</td><td>490.2</td><td rowspan="3">1581.8</td><td>477.4</td><td rowspan="3">1535.1</td><td>Aman Rice</td><td>490.2</td><td rowspan="2">1062.9</td><td>477.4</td><td rowspan="2">1025.5</td></tr>
<tr><td>Aush Rice</td><td>572.7</td><td>548.1</td><td>Aush Rice</td><td>572.7</td><td>548.1</td></tr>
<tr><td>Boro Rice</td><td>518.9</td><td>509.6</td><td>Vegetables</td><td></td><td>273.4</td><td></td><td>270.0</td></tr>
<tr><td>Turf Grass</td><td></td><td>1006.2</td><td></td><td>976.4</td><td>Turf Grass</td><td></td><td>1006.2</td><td></td><td>976.4</td></tr>
</table>

In the CROPWAT for Model 1(Rice only), rice is cultivated on 74% of the total land and rest 26% is covered with turf grass.
Using Table 6, Table 7 and Equation (5), anual recharge can be calculated for rice cultivation.
Annual Recharge (For Khulna) = 1748 – (1581.8×0.74 + 1006.2×0.26) = 316 mm
Annual Recharge (For Barisal) = 2070 – (1535.1×0.74 + 976.4×0.26) = 680 mm
Similarly, for Model 2 (Rice and vegetables), considering 45% area for rice (Aush and Aman), 29% area for small vegetables and 74% area for turf grass,
Annual Recharge (For Khulna) = 1748 – (1062.9×0.45+ 273.4×0.29 + 1006.2×0.26) = 928 mm
Annual Recharge (For Barisal) = 2070 – (1025.5×0.45+ 270.0×0.29 + 976.4×0.26) = 1276 mm

3. RESULTS AND DISCUSSION

From the CROPWAT analysis it was evident that, cultivation of rice always demands a large amount of water for its growth and maturity in different stages throughout the crop period. To justify this fact, Figure 4 illustrates the irrigation requirement for Aman Rice in different stages from land preparation to harvesting. IR and ET_c are calculated for Barisal region. The planting date is 25th of August, but the maximum amount of irrigation is required before the planting date. This is because the nursery and the land preparation (including puddling) starts 30 days before the planting date and this period needs significant volume of water. Even the presence of high amount of rainfall is not enough to reduce the amount of irrigation at this time. Farmers can provide this water using surface water irrigation. After the planting, there is enough rainfall and no irrigation is required.

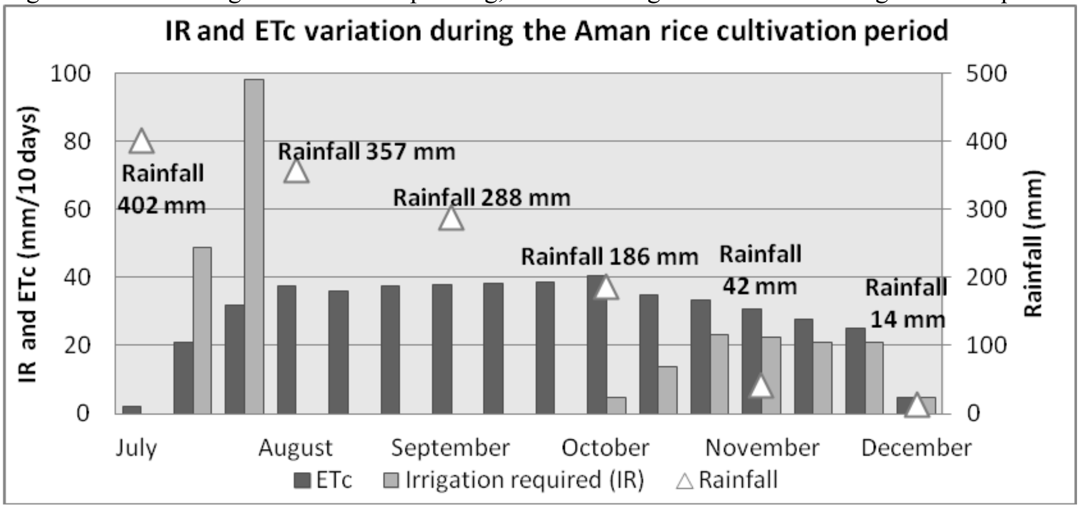

Figure 22: Variation of Evapotranspiration and Irrigation Required for Aman Rice Cultivation Period in Barisal

The variation of irrigation required over the year for rice only and rice and vegetables combination is shown in Figure 5 and Figure 6. The unit of IR mm/dec means millimeters per 10 days. Cultivating only rice throughout the whole year demands much irrigation than cultivating the combination of rice and vegetables. The peaks in the graphs represent the large amount of water requirement during the nursery and land preparation period for rice cultivation. If vegetables are cultivated during winter period (December to February), these peaks are absent. Another feature of the graphs is that the Irrigation Required (IR) is almost zero during the middle part of the year (May – August). During this time heavy rainfall occurs due to the presence of monsoon period.

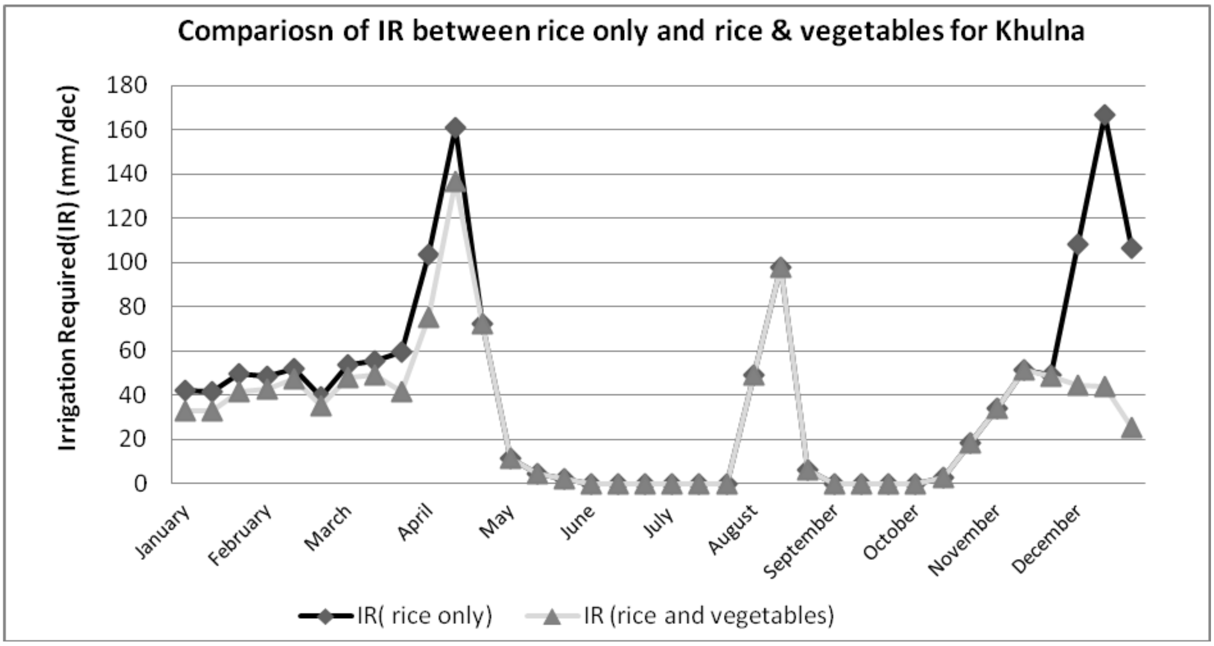

Figure 23: Variations of Irrigation Requirement for Two Different Cropping Patterns in Khulna.

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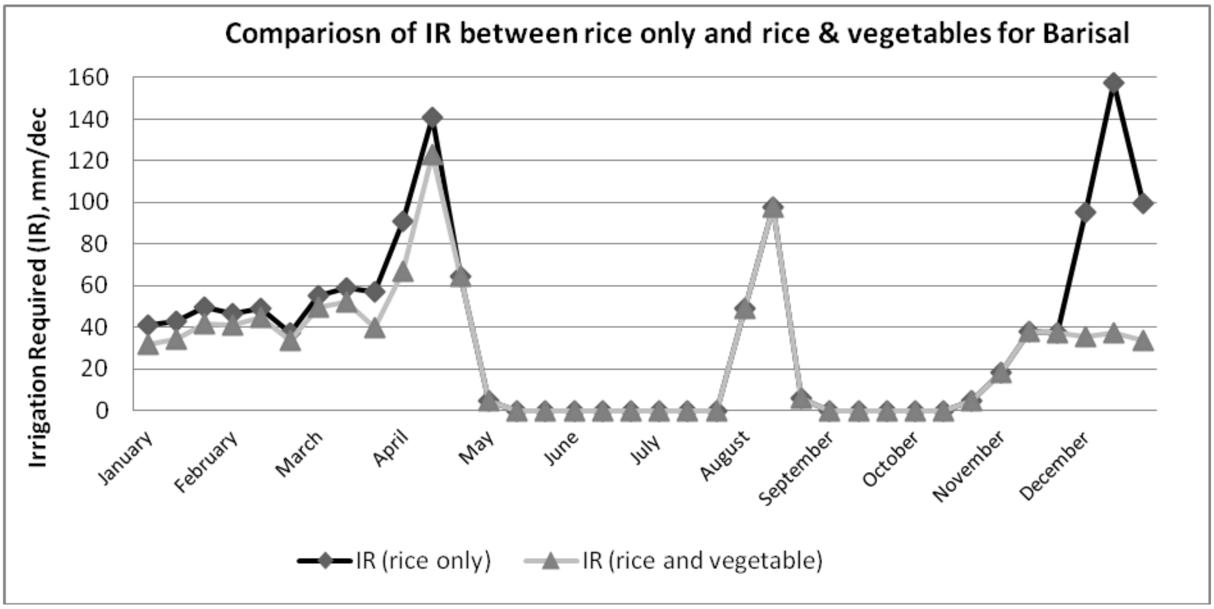

Figure 24: Variations of Irrigation Requirements for Two Different Cropping Patterns in Barisal.

As rainfall is low during the dry winter season (November to April), irrigation is dependent on groundwater extraction. In order to reduce the demand on groundwater during the dry winter period, it may be logical to switch to crop type that demands low water for its growth and maturity, i.e. small vegetables. This will also increase the annual groundwater recharge in the aquifer (Figure 7).

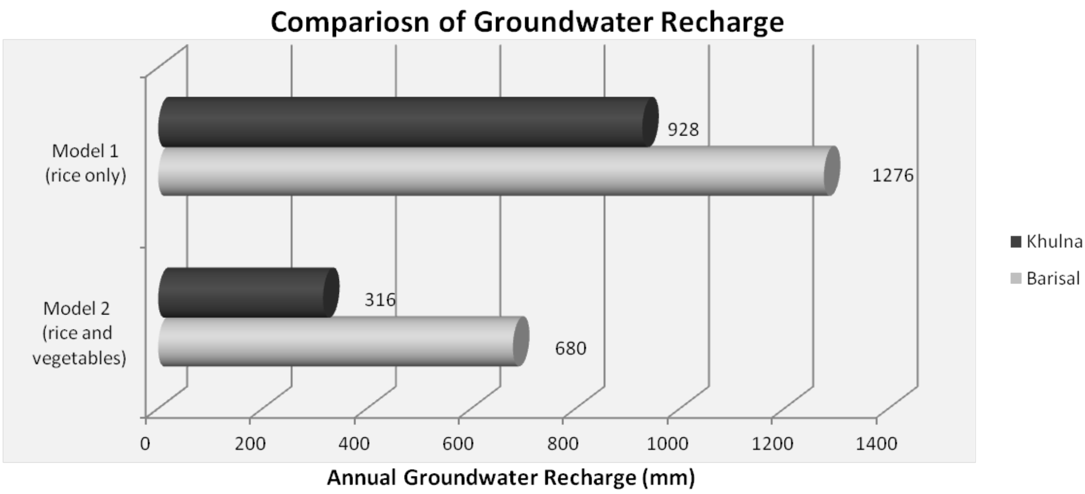

Figure 25: Comparison of Groundwater Recharge for Two Different Cropping Patterns

4. CONCLUSION

The study reveals that, in terms of groundwater extraction, it is always beneficial to cultivate crops like small vegetables instead of rice during the dry period (November to April) of the year. Situating within the coastal zone, Khulna and Barisal regions are vulnerable to problems like soil and groundwater salinity and incessant extraction of groundwater will always make the problem worse. In future, due to sea level rise, an additional degree of worsening of the problem will be added up as salt water intrusion in the coastal aquifers will occur in a rapid rate. To cope up with these upcoming threats, it is important to make necessary adjustments and adaptation in case of cultivation practice. Exploring alternating cropping options and finding the most appropriate one will be a real challenge. The methodology used in the study can be used as a guideline to explore different cropping options. The economic value of the alternative crop also needs to be considered while selecting the most appropriate cropping pattern. In addition, crop that can withstand increased salinity should be given more priority.

REFERENCES

Allen, R. G., Pereira, L. S., Raes, D. & Smith, M., 1998. *Crop evapotranspiration - Guidelines for computing crop water requirements - FAO Irrigation and drainage paper 56.* FAO - Food and Agriculture Organization of the United Nations, Rome.

Banglapedia, 2012. *National Encyclopedia of Bangladesh.* [Online] Available at: http://www.banglapedia.org/HT/F_0117.htm ; [Accessed December 2013]

Brouwer, C. & Heibloem, M., 1986. *Irrigation Water Management: Irrigation Water Needs.* FAO- Food and Agriculture Organization of the United Nations.

CZPo, 2005. *Coastal Zone Policy,* Dhaka.: Ministry of Water Resources, Government of the People's Republic of Bangladesh

FAO CLIMWAT, 2012. *FAO Water Development and management Unit.* [Online] Available at: http://www.fao.org/nr/water/infores_databases_climwat.html; [Accessed December 2013].

FAO, 2012. *FAO Water, Natural Resource and Environment Department.* [Online] Available at: http://www.fao.org/nr/water/infores_databases_cropwat.html ; [Accessed December 2013].

Haque, S., 2006. Salinity Problems and Crop Production in Coastal Regions of Bangladesh. *Pakistan Journal of Botany,* 38(5), pp. 1359-1365.

Islam, Z., 2000. Land Use Pattern in Bangladesh and Future Food Production Challenges: Are We Heading Towards a Disaster. *Bangladesh Environment, Bangladesh Rice Research Institute.*

Oryza, 2011. Bangladesh Targets 35 Million Tons of Rice for 2011-12 Crop Year – Oryza Takes a Look at Bangladeshi Production. *Oryza - The Global Marketplace for Rice Industry*, 08 December. Available at: http://oryza.com/Rice-News/13783.html.

Pender, J., 2008. Community-led adaptation in Bangladesh. *Forced Migration Riview, Issue 31*, October, pp. 54-55, Available at: http://www.fmreview.org/FMRpdfs/FMR31/FMR31.pdf

Rasid, H. & Paul, B. K., 1987. Flood problems in Bangladesh: Is there an indigenous solution. Environmental Management, *Volume 11, Number 2,* pp. 155-173.

Roy, D. C., 2011. Vulnerability and Population Displacements due to Climate-Induced Disasters in Coastal Bangladesh. Available at: http://www.ehs.unu.edu/file/get/5410

Shamsudduha, M., Chandler, R. E., R. G. T. & Ahmed, K. M., 2009. Recent trends in groundwater levels in a highly seasonal hydrological system: the Ganges-Brahmaputra-Meghna Delta. *Hydrology and Earth System Sciences,* p. 4125–4154.

Smajstrla, A. & Zazueta, F., 1996. *Estimating Crop Irrigation Requirements for Irrigation System Design and Consumptive Use Permitting.* University of Florida, IFAS Extension , AE257. Available at: http://edis.ifas.ufl.edu/ae078

Soil, 2012. *The Seed site.* [Online] Available at: http://theseedsite.co.uk/soil.html; [Accessed December 2013].

Wikipedia, 2012. *Geography of Bangladesh.* [Online] Available at: http://en.wikipedia.org/wiki/Geography_of_Bangladesh; [Accessed December 2013]

APPENDIX

Table 6: Monthly Rainfall and Evapotranspiration for Khulna and Barisal [Source: CLIMWAT 2.0]

	Khulna		Barisal	
Month	Rainfall (mm)	ET_o (mm/day)	Rainfall (mm)	ET_o (mm/day)
January	6	2.32	6	2.32
February	16	3	19	2.99
March	46	4.12	41	4.01
April	88	4.57	106	4.4
May	178	4.35	209	4.04
June	357	3.26	400	3.13
July	353	3.31	402	3.2
August	330	3.07	357	2.97
September	217	3.51	288	3.4
October	130	3.47	186	3.4
November	22	3.03	42	2.94
December	5	2.42	14	2.41
	Total 1748.0 mm	Average 3.37 mm/day	Total 2070.0 mm	Average 3.27 mm/day

Table 7: Crop Properties of Rice in Different Stages [Extracted from CROPWAT 8.0].

				Transplantation to Harvesting Period				
	Nursery	Land Preparation		Growth Stage				
Stage		Total	Puddling	Initial	Develop	Mid season	Late season	Total
Length (days)	30	20	5	20	30	40	30	150
K_c (dry)	0.70	0.30		0.50		1.05	0.70	
K_c (wet)	1.20	1.05		1.10		1.20	1.05	
Rooting depth(m)				0.10		0.60	0.60	
Puddling depth(m)			0.40					
Nursery area (%)	10.0							
Critical depletion (m)	0.20			0.20		0.20	0.20	
Yield response factor				1.00	1.09	1.09	1.09	
Crop height (m)						1.00		

Table 8: Crop Properties of Small Vegetables in Different Stages [Extracted from CROPWAT 8.0].

	Transplantation to Harvesting Period				
Stage	Initial	Develop	Mid season	Late season	Total
Length (days)	20	30	30	15	95
K_c	0.70		1.05	0.95	
Rooting depth(m)	0.25		0.60	0.60	
Critical depletion (m)	0.30		0.45	0.50	
Yield response factor	0.80	0.40	1.20	1.00	

RESPONSE OF COASTAL STRUCTURES DUE TO WAVE LOADING

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ABSTRACT:

Bangladesh has always been a disaster prone country. Among these disasters cyclone is a nightmare for the coastal districts. A large number of districts in the coastal areas are mostly affected by these cyclones. These cyclones cannot be stopped but human lives and properties can be saved by taking appropriate precautions. Proper design guidelines need to be followed in building structures in the coastal region, but there is lack of proper design guide for building in the coastal regions of Bangladesh. The structural analysis procedure for these structures subjected to ocean wave loading is not very rigorous. Therefore, a lack of proper guidance seems absent even at then topmost level of engineering judgment. Even the approach suggested in the Bangladesh National Building Code (BNBC 1993) seems quite casual. The main objective of the present work is to implement and study the effect of hydrodynamic wave loading derived from cyclones in the coastal structure; i.e., to study the structural behavior of various types of buildings to ocean wave loading. In order to study the effect of cyclones on multi-storied coastal structures, four different structures (3-, 6-, 12- and 24-storied buildings) are used for nonlinear dynamic analysis. After applying wave, current and wind forces, their deflections vs. curvature relationships are simulated numerically. The effect of wind is found to be extremely significant for the 6-, 12- and 24-storied buildings but not so much for the 3-storied building. Both the deflections and maximum curvatures of the RC structures are found to be well beyond the allowable safe and serviceability limits, rendering the structures to failure.

Keywords:*Natural Hazards, Wave, Wind, Coastal Structures, Nonlinear Dynamic Analysis.*

1. INTRODUCTION

Bangladesh has always been a disaster prone country. Almost every year various kinds of disasters like excessive rainfall, flood, cyclone, and drought affect the country. Among these disasters cyclone is a nightmare for the coastal districts. A large number of districts in the coastal areas are mostly affected by these cyclones. The Bay of Bengal is the breeding ground for tropical cyclones and Bangladesh is the worst victim in terms of fatalities and economic losses incurred. In the last 100 years 508 cyclones have originated in the Bay of Bengal, 17 percent of those have hit Bangladesh, amounting to a severe cyclone almost once every three years. Every cyclone leaves behind a huge damage, including lives, shelters and other properties. The majority of them have claimed more than five thousand lives each. These cyclones cannot be stopped but human lives and properties can be saved by taking appropriate precautions. Proper design guidelines need to be followed in building structures in the coastal region, but there is lack of proper design guide for building in the coastal regions of Bangladesh.

These cyclones cause damage to coastal buildings. In the coastal regions, usually, poor people cannot afford a concrete building, which may sustain in the high wind and wave forces. These buildings are usually made of low cost local materials without any engineering knowledge. Life of coastal people may change a lot if engineering knowledge can be used to improve the designs and materials. The country in general including the rich class, the structural designers and decision makers seem to be extremely insensitive and quite oblivious of their suffering. Technocrats do not adequately support housing project for low-income and flood vulnerable communities. The usual tendency is to apply the same model irrespective of context. A comparatively new trend is the development of tourist facilities (including hotels, restaurants) along the coastlines. The threat of cyclone-induced loads is quite real on these multi-storied structures.

Fig. 1: Devastation on the coastal structures after Cyclone Sidr

In addition to the cyclones, tsunamis present another threat looming large on the coastal structures. With the increased threat of earthquakes in Bangladesh, the risk of tsunamis poses a new threat that had never been encountered even in this disaster-prone country. Though the generation sources between tsunami and the storm surge are different, the physical characteristics of their wave propagation (in deep water), nonlinear transformation (in shallow water) and run-up (land inundation) are identical. Analytical and numerical models for each of these phases have been developed. Numerical codes for storm surge SSM (Storm Surge Model, Mastenbroek*et al*. 1993), wave in open ocean WAM (WAve Model, Komen *et al*. 1994), coastal wave transformation SWAN (Simulating WAveNearshore, Booij*et al*. 1999), and surf-zone processes and run-up COULWAVE (Cornell University Long WAVE, Lynette *et al*. 2002) are representative examples. These models have been coupled together to form a source to shore water elevation and inundation prediction for emergency management in the state of Hawaii (Cheung, et al. 2003).

The structural analysis procedure for these structures subjected to ocean wave loading is not very rigorous. Even the approach suggested in the Bangladesh National Building Code (BNBC 1993) seems quite casual, using language like *'Required loading shall be determined in accordance with the established principles of mechanics'* and *'The hydrodynamic load applied on a structural element due to wind induced local waves of water, shall be determined by a rational analysis using an established method'*, without elaborating on that. Therefore, a lack of proper guidance seems absent even at the topmost level of engineering judgment. The importance of this work can be justified in this context.

The main objective of the work is to implement and study the effect of hydrodynamic wave loading derived from cyclones in the coastal structure; i.e., to study the structural behavior of buildings various types of buildings to ocean wave loading. The more specific objectives is to analyze and design of four model RC buildings of different heights (3-,6-, 12- and24-storied), considering vertical loads only, using the structural analysis software ETABS andderived the moment-curvature (M-ϕ) relationship of individual beam and column sections and finally,analyzed the structures using numerical methods based on a rigorous nonlinear model of the material, also considering dynamic characteristics of wave and wind load.

2. LOADS ON COASTAL STRUCTURES

Various loads affect on structures like dead load, live load, wind load, earthquake load, wave load, current load etc. In the coastal region besides DL, LL, wave load from storm surges and wind load are much effective. In present work, we have given much importance on loads from storm surges and wind loads.

2.1 Wind Load

Wind is a significant component of cyclone loading and is very important for the analysis and design of a large number of structures. It governs the design of several structures, particularly those located in the open area or close to the seashore; e.g., marine and offshore structures as well as cyclone shelters and structures under small dead loads like industrial trusses.

The wind load on a structure can be obtained by integrating the dynamic wind pressure over the surface it acts on. The velocity-squared term of Bernoulli's pressure equation is considered most important, and thus the dynamic pressure is approximated by

$$p_d \cong \rho_{air} V^2/2 \qquad (1)$$

Where V is the wind velocity and ρ_{air} the mass density of air. The wind-force is therefore the integral of this pressure over the structure's surface-area. If $W(z)$ is the width of the structure at any point, then the total wind force is given by

$$F_w(t) \cong \int \rho_{air}(z,t)\}^2/2\ W(z)\ dz \qquad (2)$$

Here $V(z,t)$ and $W(z)$ are usually functions of z, but ρ_{air} expected to remain constant. The mean wind velocity $\bar{V}$ can be expressed as a function of height above the terrain.

2.2 Wave Loading

Waves can affect coastal structures in a number of ways. The most severe damage is caused by breaking waves. The force created by waves breaking against a vertical surface is often ten or more times higher than the force created by high winds during a storm event. Waves are particularly damaging due to their cyclic nature and resulting repetitive loading. Because typical wave periods during hurricanes range from about 6 to 12 seconds, a structure can be exposed to 300 to 600 waves per hour, resulting in possibly several thousand load cycles over the duration of the storm.

2.2.1 Wave Kinematics by Linear Wave Theory (LWT)

The wave elevation η from the linear wave theory is given by the summation of a number of sinusoidal waves;

i.e., $\eta = \sum a_i \cos(k_i\, x - \omega_i t + \theta_i)$ (3)

with $\omega_i = 2\pi f_i = 2\pi/T_i$, $k_i = 2\pi/L_i$

where, ω_i is the frequency of the i^{th} wave in radians per second, f_i is the frequency of the i^{th} wave in Hertz (Hz), T_i is the i^{th} wave period, k_i is the i^{th} wave number, L_i is the i^{th} wave-length, θ_i is a randomly generated phase angle of the i^{th} wave. Using the linear wave theory,

Horizontal wave-velocity, $u = \sum a_i \omega_i [\cosh (k_i(z+d))/\sinh k_i d] \cos(k_i\, x - \omega_i t + \theta_i)$ (4)

Vertical velocity, $w = \sum a_i \omega_i [\sinh (k_i(z+d))/\sinh k_i d] \sin(k_i\, x - \omega_i t + \theta_i)$ ……...(5)

Horizontal acceleration, $\partial u/\partial t = \sum a_i \omega_i^2 [\cosh (k_i(z+d))/\sinh k_i d] \sin(k_i\, x - \omega_i t + \theta_i)$ (6)

2.2.2 Modifications to Linear Wave Theory

Linear wave theory predicts wave kinematics only up to the mean water level (MWL). To extend it to the free surface, two types of modifications are generally applied. Both of these are based on the wave kinematics at the MWL, and involve modifications to the hyperbolic functions describing the kinematics.

Extrapolation: Here, the kinematic properties at the MWL are retained, and those at the free surface are predicted by some approximate functional expansions about this value. Three types of extrapolations are generally used Hyperbolic – Here, the wave kinematics between the MWL and free-surface are assumed to follow the same hyperbolic variations with depth as they do up to the MWL; Linear – The wave kinematics up to the MWL are extended linearly up to the free-surface following the trends at the MWL. That is, the hyperbolic functions are expanded in Taylor's series, retaining the first terms only;

Uniform – The wave kinematics at the free surface are the same as the ones at the MWL;

i.e., $\cosh (k(z+d))$ [for $0 \le z \le \eta$] $\cong \cosh (kd)$ (7)

Stretching: Extrapolations are consistently found to over-estimate the wave kinematics, particularly near the free surface. An alternative is to assume the kinematics at the MWL to be applicable at the free surface, and use the hyperbolic variation with depth for $-d \leq z \leq \eta$.. Two types of stretching have been suggested, Wheeler (1969) and Chakrabarti (1971)

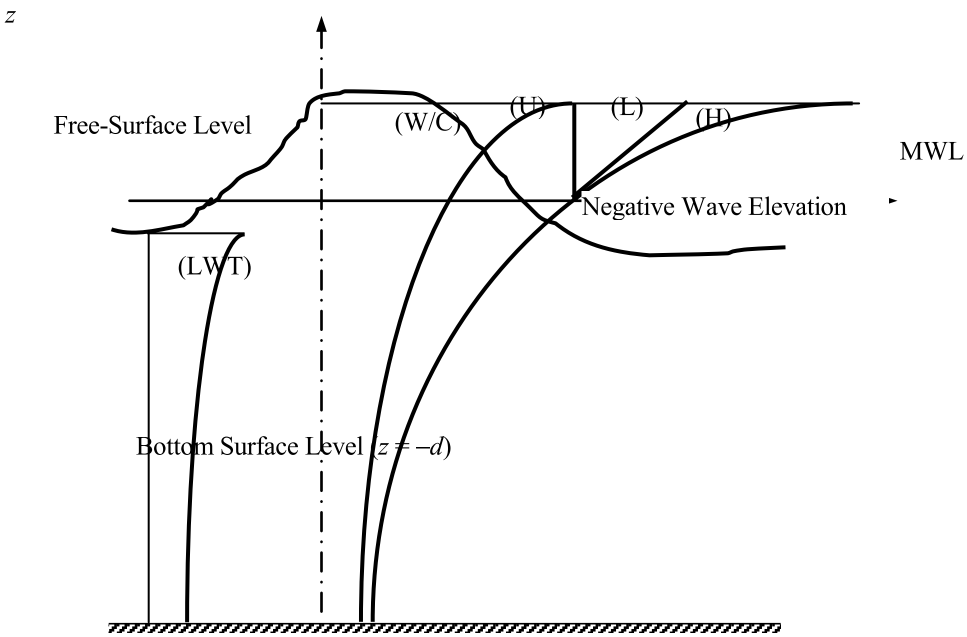


Fig. 2: Different Nonlinear Wave Kinematics Approximations [(H): Hyperbolic Extrapolation, (L): Linear Extrapolation, (U): Uniform Extrapolation(W/C): Wheeler/Chakrabarti Stretching]

2.2.3 Wave Spectra

Various idealized spectra are used in oceanography and ocean engineering. The simplest one was proposed by Pierson and Moskowitz in 1964. They assumed that if the wind blew steadily for a long time over a large area, the waves would come into equilibrium with the wind.
The Pierson-Moskowitz equation is

$$S(\omega) = (\alpha\, g^2/\omega^5)\exp\{-\beta(\omega_0/\omega)^4\} \qquad (8)$$

where, $\omega = 2\pi f$, f = wave-frequency in Hertz, $\alpha = 8.1 \times 10^{-3}$, $\beta = 0.74$, $\omega_0 = g/U_{19.5}$ where $U_{19.5}$ is the wind speed at a height of 19.5m above the sea surface, the height of the anemometers. For most air flow over the sea the atmospheric boundary layer has nearly neutral stability, and

$$U_{19.5} \cong 1.026\, U_{10} \qquad (9)$$

assuming a drag coefficient of 1.3×10^{-3}. The frequency of the peak of the Pierson-Moskowitz spectrum is calculated to be

$$\omega_p = 0.877g/U_{19.5} \qquad (10)$$

The significant wave-height is calculated to be

$$H_{1/3} = 0.21\,(U_{19.5})^2/g \qquad (11)$$

After analyzing data collected during the Joint North Sea Wave Observation Project (JONSWAP), it was found that the wave spectrum is never fully developed. It continues to develop through nonlinear, wave-wave interactions even for very long times and distances. Hence an extra and somewhat artificial factor was added to the Pierson-Moskowitz spectrum in order to improve the fit to their measurements. The JONSWAP spectrum is thus a Pierson-Moskowitz spectrum multiplied by an extra peak enhancement factor γ^r

$$S(\omega) = (\alpha\, g^2/\omega^5)\exp\{-5/4(\omega_p/\omega)^4\}\ \gamma^r \qquad (12)$$

where $r = \exp[-(\omega^2-\omega_p)^2/2\sigma^2\,\omega_p^{\,2}]$

2.2.4 Morison's Equation

Morison's equation is easily the most used equation for calculation of hydrodynamic force on marine/offshore structures. For slender structures, it is often considered as valid as the more rigorous diffraction theory, and has been used in several practical as well as research works.

According to the (modified) Morison's equation, the horizontal wave-force and moment on a differential vertical segment dz are given (after adjustments for structural acceleration) by

$$dF_x = [K_I a_x + K_D|u_r|u + K_m u_r \partial w/\partial z]\, dz \text{ ; and } dM_{y0} = z\, dF_x \tag{13}$$

where $K_I = \rho C_I A$, $K_D = \rho C_D R$, $K_m = \rho C_m A$,
[ρ = Water density, C_I = Inertia coefficient, C_D = Drag coefficient, $C_m = C_I - 1$,
A = Cross-sectional area, R = Radius = Half-width of projected surface]
a_x = Horizontal wave-acceleration = $du/dt = \partial u/\partial t + u\, \partial u/\partial x + w\, \partial u/\partial z$,
u_r = Relative horizontal velocity = Horizontal (wave – structural) velocity.

In Eq. (13), the first term gives the inertia force, the second is the drag and the third term corresponds to the axial-divergence term. Total horizontal force $F_x = \int dF_x$ and moment about bottom of the structure is $M_{y0} = \int z\, dF_x$, where $\int$ implies integration between $z = -L$ (bottom of the structure) and $z = \eta$ (instantaneous wave-elevation).

2.3 Current Loading

In addition to the inertia and drag forces, a mean drift force acts on the structure that cannot be predicted by Morison's equation. However the drag term in Morison's equation can account for the force due to uniform current of velocity U_c, the force being given by

$$F_c = K_D U_c^2 L \tag{14}$$

2.4 Flood Loads on Structures at Inland Areas

For structures sited at inland areas subject to flood, loads due to flood shall be determined considering hydrostatic effects which shall be calculated based on the flood elevation of 50-year return period. For riverside structures, hydrodynamic forces, arising due to approaching wind-generated waves shall also be determined in addition to the hydrostatic load on them. In this case, the amplitude of such wind-induced water waves shall be obtained from site-specific data.

2.5 Flood and Surge Loads on Structures at Coastal Areas

For structures sited at coastal areas, the hydrostatic and hydrodynamic loads shall be determined as follows

2.5.1 Hydrostatic Loads

The hydrostatic loads on structural elements and foundations shall be determined based on the maximum static height of water, H_m produced by floods or surges as given by

$$H_m = \text{Max}(h_s, h_f) \tag{15}$$

where $h_f = y_T - y_g$
h_s = Maximum surge height as specified in (i) below
y_T = Elevation of the extreme surface water level corresponding to a T-year return period specified in (ii) below, meters
y_g = Elevation of ground level at site, meters.
(i) Maximum Surge Height, h_s: The maximum surge height, h_s, associated with cyclones, shall be that corresponding to a 50-year or a 100-year return period as may be applicable, based on site specific analysis. In the absence of a more rigorous site-specific analysis, the following relation may be used:

$$h_s = h_T - (x - 1)\, k \tag{16}$$

where h_T = Design surge height corresponding to a return period of T-years at sea coast, in meters, given in Table. 1 (i.e., Table 6.2.28 of BNBC'93).
x = Distance (km) of the structure site measured from the spring tide high-water limit on the sea coast, in km; $x = 1$, if $x < 1$.

k = Rate of decrease in surge height in m/km; the value of k may be taken as 1/2 for Chittagong-Cox's Bazar-Teknaf coast and as 1/3 for other coastal areas

(ii) Extreme Surface Water Level, y_T: The elevation of the extreme surface water level, y_T for a site during monsoon, which may not be associated with a cyclonic storm surge, shall be that obtained from a site-specific analysis corresponding to a 50-year or a 100-year return period. Values of y_T are also given in Table 6.2.29 of BNBC'93 for selected coastal locations to be used in the absence of any site-specific data.

2.5.2 Hydrodynamic Loads

The hydrodynamic load applied on a structural element due to wind-induced local waves of water, shall be determined by a rational analysis using an established method and based on site-specific data. In the absence of a site-specific data the amplitude of the local wave, to be used in the rational analysis, shall be taken as $h_w = h_s/4 \geq 1$-m. Such forces shall be calculated based on 50-year or 100-year return period of flood or surge. The corresponding wind velocities shall be 260 km/h or 289 km/h respectively.

Table 1: Design Surge Heights at the Sea Coast, h_T

Coastal Region	Surge Height at the Sea Coast, h_T(m)	
	***T* = 50-year**	***T* = 100-year**
Teknaf to Cox's Bazar	4.5	5.8
Chakaria to Anwara, and Maheshkhali-Kutubdia Islands	7.1	8.6
Chittagong to Noakhali	7.9	9.6
Sandwip, Hatiya and all islands in this region	7.9	9.6
Bhola to Barguna	6.2	7.7
Sarankhola to Shyamnagar	5.3	6.4

3. STRUCTURAL AND MATERIAL PROPERTIES

3.1 Structural Models

In order to study the effect of cyclones on multi-storied coastal structures, four different structures (3-, 6-, 12- and 24-storied building) are used for structural analysis. Similar beam size (10″× 20″) is used in all three buildings.

But the same size of columns is not used; i.e., column sizes are changed at every 6 stories. Three different sizes of columns (Center, Edge and Corner) are designed for every 6 stories. The models used are 60-ft (3 bays of 20′ each) in the long direction and 40-ft (2 bays of 20′ each) in the short direction. Uniform story height of 10-ft is used for all the buildings. The structural models are shown in the Fig. 3(typical floor plan) and Fig. 4 (typical elevation).

3.2 Material Properties

Two types of materials are used for the structures used in this study; i.e., timber for the trees (protective vegetation), concrete and steel for the multi-storied RC buildings. The structural members are designed for vertical loads only, and no special detailing/confinement is used in the design. Therefore, the ultimate strain of confined concrete is assigned as such; i.e., from conventional design provisions.

Table 2: Mechanical Properties of Materials used

Materials	Properties	Values
Concrete	Compressive strength, f_c'	3.5 ksi
	Modulus of Elasticity	3000 ksi
	Ultimate Strain (unconfined) Ultimate Strain (confined)	0.0045 0.0055~0084
Steel	Yield strength, f_y	60 ksi
	Modulus of elasticity	29000 ksi
	Ultimate Strain	0.20

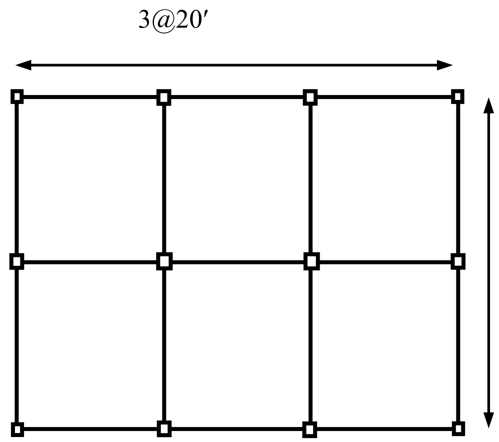


Fig. 3: Building Lay-out Plan

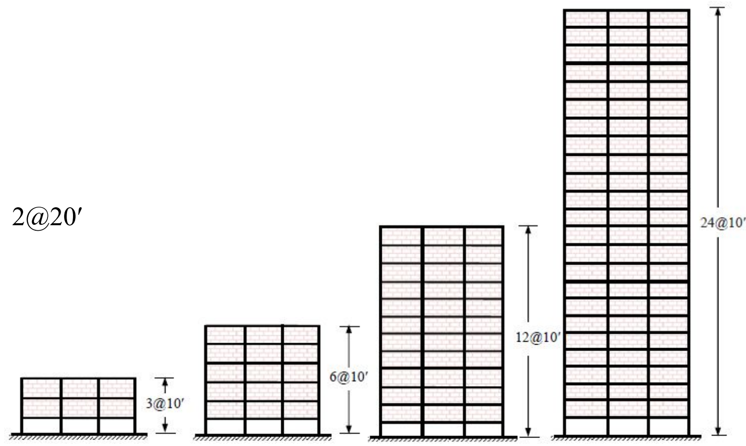


Fig. 4: Elevation of the 3-, 6-, 12- and 24-storied buildings (long direction)

3.3 Loading Condition

Structural analysis of the RC buildings involves more realistic simulation of the design cyclones, including storm surge, wave, current and wind. In view of the wide range of structural heights, the governing parameters are fixed at a storm surge of 20-ft, wave height of 6-ft, wave current velocity of 3 ft/sec and basic wind speed of 150 mph.

4. RESULTS FROM NUMERICAL ANALYSIS

4.1 Moment-Curvature Relationships of Columns

The Moment vs. Curvature (M-ϕ) relationships for center column sections of the 3-, 6-, 12- and 24-storied buildings are shown in Figs. 5(a)~6(b)

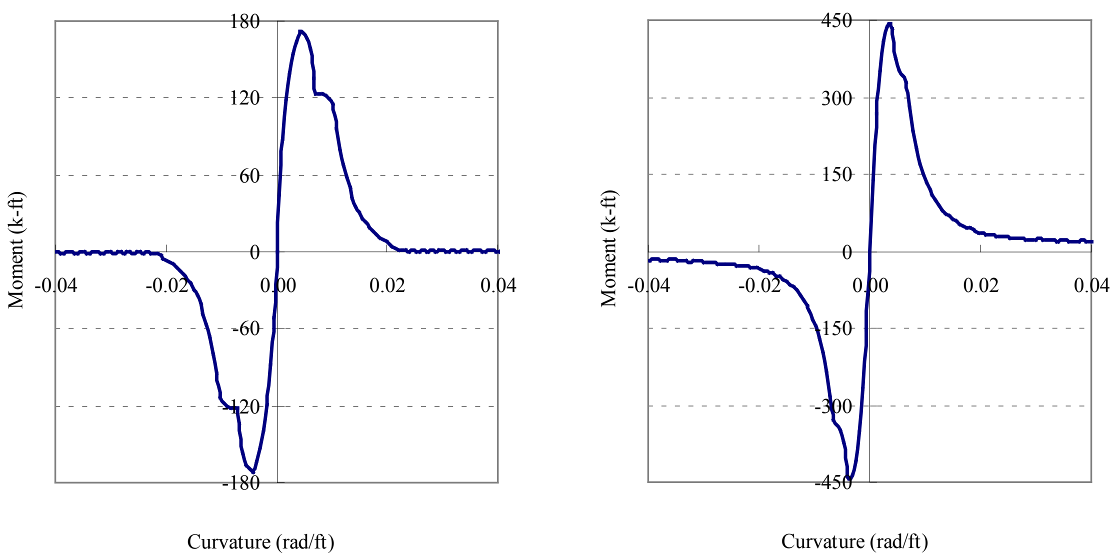


Fig. 5(a): Moment vs. Curvature of center column for 3-storied building

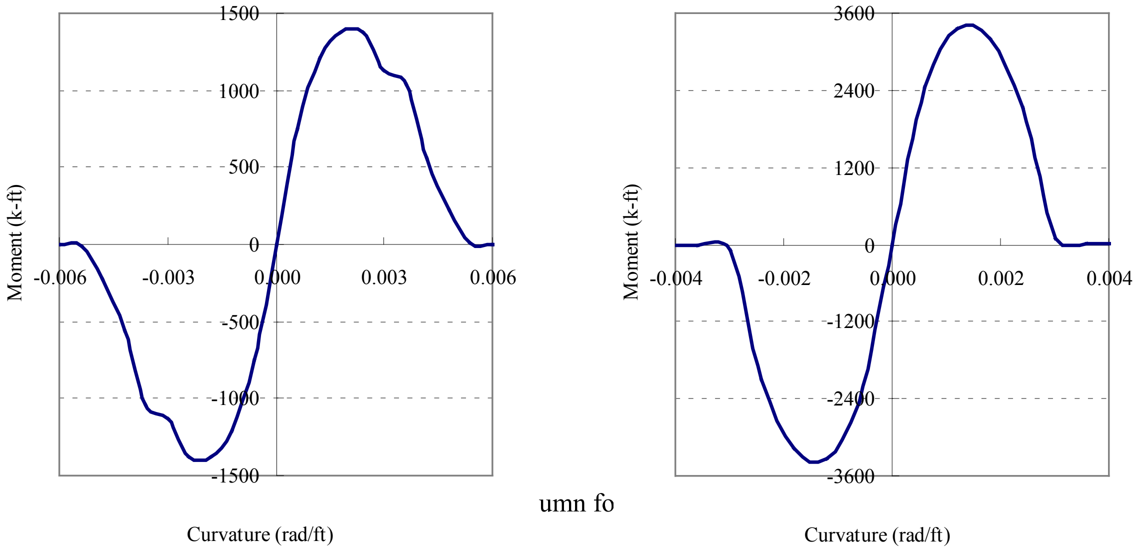
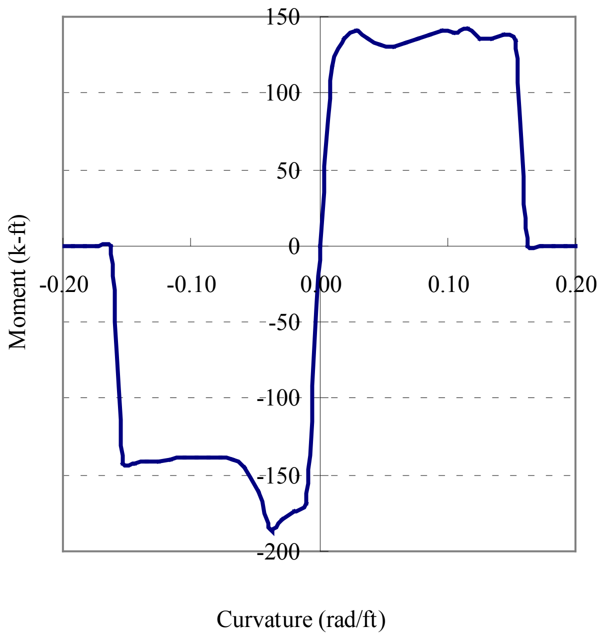
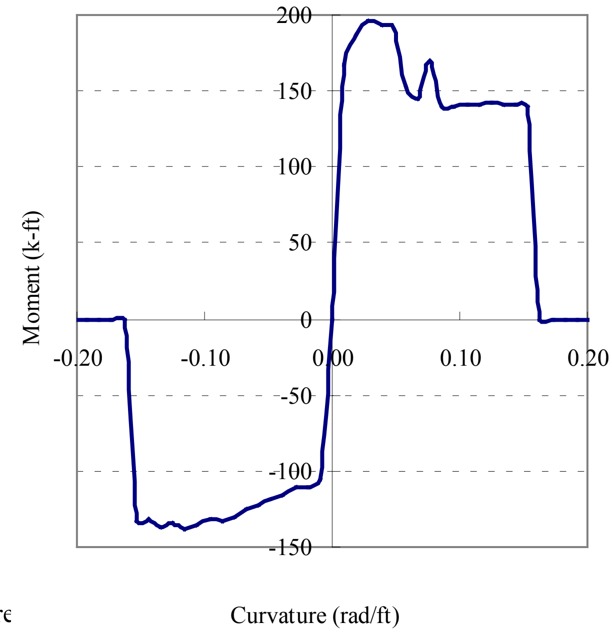


Fig. 5(b): Moment vs. Curvature of center column for 6-storied building

4.2 Moment-Curvature Relationships of Beams

Figs. 7 and 8 show the M-ϕ relationships for all the beam sections in the long and short directions, which remain identical for all the structures considered here. Moreover, each figure here has two plots, showing the corresponding relations at the end and middle section of the beams.





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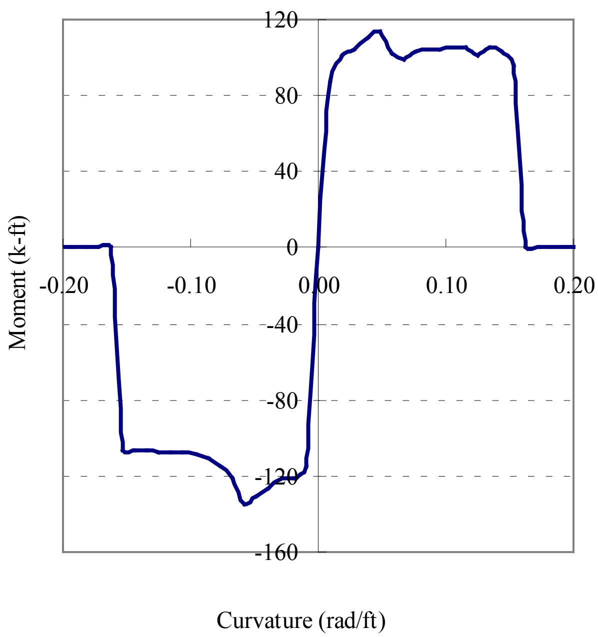

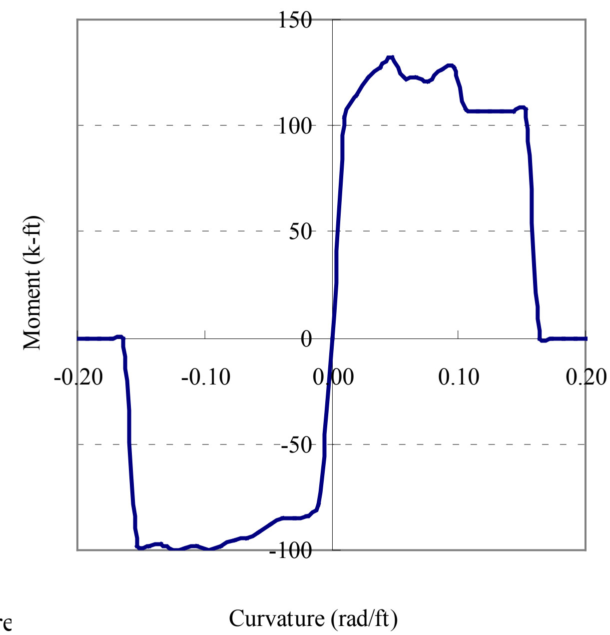

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5.1 Curvatures for Wave, Current and Wind Loading

The variation of ground floor column curvatures [Figs. 9(a)~10(b)]also demonstrates the very significant effect of the wind loads and also shows that the designed structures not only fail the serviceability criteria but also are not safe. The very large curvature requirements often far exceed the ultimate curvature capacities of the sections (shown earlier by the *M-ϕ* relationships).Each figure here has three plots, showing the corresponding relations for wave loads only (W), combined wave-current loads (WC) and wave-current-wind loads (WCW).

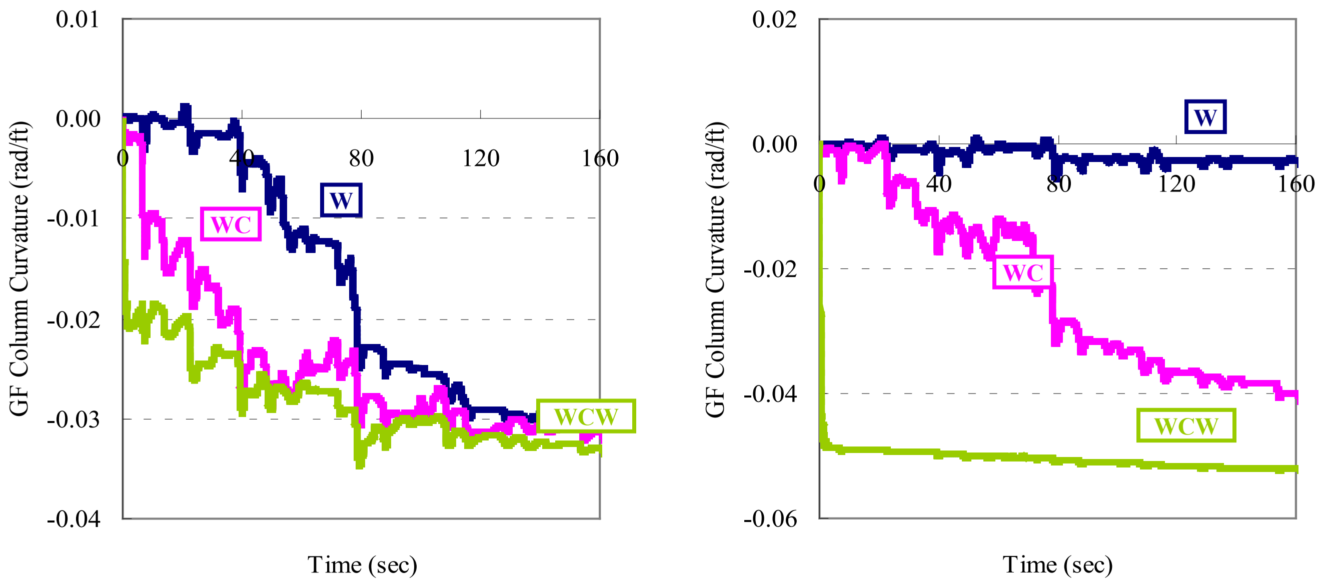

Fig. 9: Ground Floor Column Curvatures of (a) 3-Storied Building, (b) 6-Storied Building

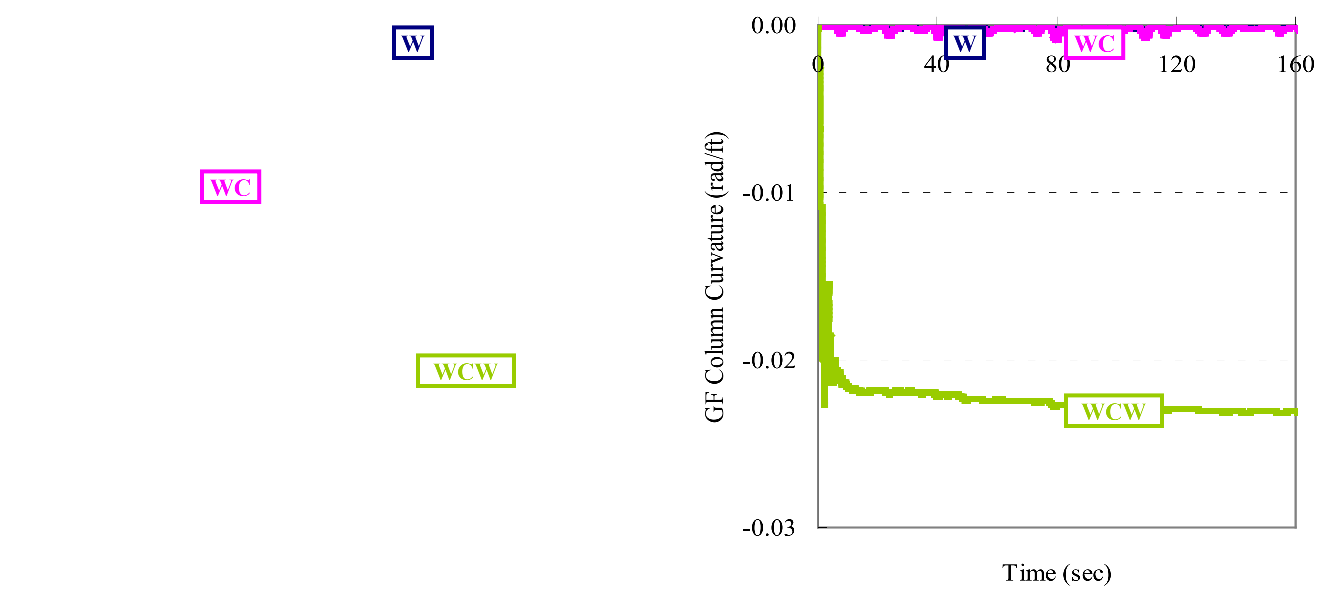

Fig. 10: Ground Floor Column Curvatures of (a) 12-Storied Building, (b) 24-Storied Building

4.4 Deflections for Wave, Current and Wind Loading

Figs. 11(a)~12(b) show the top floor deflections of the buildings for wave, combined wave-current and wave-current-wind loads [Fig. 11 for 3- and 6-storied while Fig. 12 for 12- and 24-storied building].

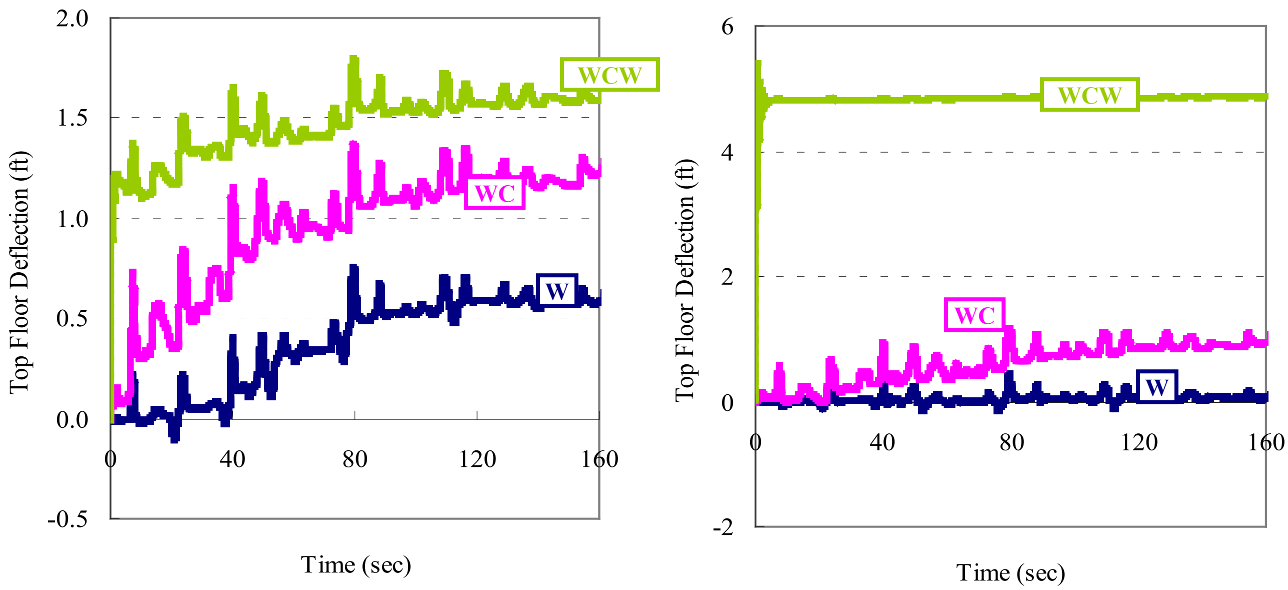

Fig. 11: Top Floor Deflections of (a) 3-Storied Building, (b) 6-Storied Building

Fig.12: Top Floor Deflections of (a) 12-Storied Building, (b) 24-Storied Building

Due to the action of the wind loads being close to the tip of the structures, leading to more significant effects on the structural deflections. Such large deflections are definitely not permissible for the structures, and alternative structural systems (e.g., larger beam and column sections, shear walls, bracings, etc.) are to be used to reduce the deflections and improve serviceability of the buildings.

Therefore, in addition to the structural measures suggested to reduce deflections, the sectional ductility need to improved using special detailing measures.

6. CONCLUSIONS

The study shows that both the deflections and maximum curvatures of the RC structures are well beyond the allowable safe and serviceability limits, rendering the structures to structural failures. Wind loads significantly increase the mean drift and push the structural vibrations to the nonlinear range. The effect of wind is found to be extremely significant for the 6-, 12- and 24-storied buildings but not so much for the 3-storied building. The effect is more pronounced for taller structures because winds act near the tip of the structures and cause more deflections and internal strains/curvatures. The main effect of current is to provide a significant mean drift of the structures, which would otherwise be absent for wave loading only.

Further studies can be done in this area considering actual cyclone data to simulate more real picture. Structural performance of buildings with shear walls, diagonal bracings and seismic detailing can also be studied under these loading. Structural models can also be tested in the laboratory under wave loading.

REFERENCES

Government of Bangladesh (GoB) (2008). "Cyclone Sidr in Bangladesh: Damage, Loss, and Needs Assessment for Disaster Recovery and Reconstruction." aReport Prepared by the Government of Bangladesh Assisted by the International Development Community with Financial Support from the European Commission.

Hossain, M. Z., Islam, M. T., Sakai, T, and Ishida, M. (2008)."Impact of Tropical Cyclones on Rural Infrastructures in Bangladesh". Agricultural Engineering International: the CIGR Ejournal.2.

Navarre Beach Fishing Pier Design (2006),"Wave Load Calculations, Methodology & Results"

Karim, N. (1999). "Options for Cyclone Protection: Bangladesh Context" Department of Sociology, University of Dhaka.

Rahman, M. M., and Biswas, S. K. (2009)."Feasible Solution of Protection and Adaptation Strategy for Coastal Zone of Bangladesh." Pakistan Journal of Meteorology, 8,9-19.

Reddy, S.R.K and Reddy, G.C.S. (2008)"Response of Coastal Structures against Seismic and Tsunami Forces" Proceedings of the Eighteenth International Offshore and Polar Engineering Conference Vancouver, BC, Canada, July 6-11, 2008

Saleken, M. (2011)."Response of Low Cost Structure to Wind Load." B.Sc. Engrg. Thesis, Dept of Civil Engrg., UAP.

SUSTAINABLE NAVIGABILITY IMPROVEMENT OF MONGLA PORT

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ABSTRACT

The aim of the study was to formulate different engineering options for short and long term solutions for the improvement of navigability of Mongla Port by applying latest technology of numerical models associated with sophisticated field survey. A number of techniques were used for assessing sedimentation and erosion rates in the estuarine channel. These include analysis of existing hydrometric and bathymetric data and hydraulic modelling. A suite of one-dimensional and two-dimensional models more focussed on the study purpose has been developed for the South-west region, for the Pussur-Sibsa river system and also for Bay of Bengal. Combination of several engineering interventions like dredging, constricting the channel and at places entraining the flooding and ebbing channels in a single channel by guide bundhs and tidal basins at suitable location have been studied. The environmental and social impacts have been evaluated for every engineering intervention. At the last is has found that, the tidal river management in combination with canalization is technically feasible, environment friendly, economically viable and socially acceptable solution. This option has recommended for implementation as a long-term solution for maintaining a sustainable navigable channel.This Paper identifies the main causes of Navigation Problem of Mongla port and recommended a long term sustainable solution.

Keywords: *Mongla Port, Pussur River, Navigation, Dredging, Outer Bar*

1. INTRODUCTION

Mongla Port is the second largest sea port of Bangladesh. It is situated on the east bank of Pussur River, approximately 131 Km upstream from the Fairway buoy (approaches to the Pussur from Bay of Bengal). The navigability of the Mongla Port is vital to the economy of the country. In 1950, the port started as Chalna Anchorage at Chalna, but due to navigability problem the port was not sustainable at that location. In 1964, the site for the berths in Mongla Port, was selected on the left bank of the Pussur River, for an average 8.5 m draft ship. Since 1970, after the construction of Mongla Port Jetty, the depths in the navigation channel reduced significantly and regular maintenance dredging is required to provide adequate depth alongside the berths, in the approaches to the berths and in the Southern Anchorage areas. From the development of Port, several dredging efforts had been made to maintain the navigability of the Pussur River. But none of dredging efforts could sustain a navigable channel, because of continued, high siltation rates.

It is evident that the morphological behavior of the river system, sedimentation and erosion pattern of the Pussur River is frequently changing. While it is of extreme importance in connection with safe and economic handling of sea going vessel, it however encounters uncertainty due to continuous and high siltation. In the total channel, two areas are very sensitive for navigation. One is about 13 km channel, started from port jetty known as Harbour Area and another is outer bar. In the recent years water depths in the Harbour Area has reduced significantly even upto -4.0 m CD (chart datum). The outer bar is relatively stable with sea bed elevation of -6.4 m CD (chart datum). With the existing depth in the outer bar, maximum 8.5 m draft vessel can cross the outer bar and enter the port at normal high tide. But the depths over the anchorage area of the channel permit anchoring of more than 9.0 m draft vessels. To find a sustainable solution of this problems Mongla Port Authority has conducted an in depth study in association with the Institute of Water Modelling (IWM) titled "Feasibility study for the improvement of navigability of Mongla Port". In this paper, the main navigation problem of Mongla Port has identified. Simultaneously possible and economically viable solutions of identified problems has suggested.

2. METHODOLOGY

A number of techniques are in practice for assessing sedimentation and erosion rates in the estuarine channel. These include analysis of existing hydrometric and bathymetric data and hydraulic modelling. Considering the changes in the Pussur river being the outcome of intervention in the upstream basin and the regional development and natural coastal and deltaic processes and the outer bar being influenced by the huge tidal dynamics of Bay of Bengal, in that context, numerical model is found to be the best tool to handle such interaction. Existing river situation being extremely important for establishing the initial condition on a model, it has been developed by collecting huge amount of bathymetric and hydrometric data. A suite of one-dimensional and two-dimensional models more focussed on the study purpose has been developed for the South-west region, for the Pussur-Sibsa river system and also for Bay of Bengal. Combination of several engineering interventions like dredging (intelligently selected alignments and cross-sections), constricting the channel and at places entraining the flooding and ebbing channels in a single channel by guide bundhs and tidal basins at suitable location have been studied in improving the navigability of Mongla Port. The environmental and social impacts have been evaluated for every engineering intervention.

3. NAVIGATION PROBLEM IN THE PUSSUR RIVER

The commercially important portion of the Pussur river begins at Chalna and extends about 80 km up to Akram Point and then up to Fairway Buoy which add another 70 km length of navigation channel. The maximum draft of vessels that can enter the port jetties varies between 6.0m and 7.0m, depending on the tide and weather conditions. This requires a minimum bed level of –6 m CD. The available bed level for navigation as based on the measured bed level during March 2011 of the Pussur river from Jetty to Akram point is shown in Figure 2. The bed level shows that from Maidara to Joymonirgoal the required navigation channel does not exist. On the other hand, the river maintains a good navigable channel from Joymonirgoal to Hiron Point which is sufficient even for 9 m draft vessel with minimum dredging at a few locations. The channel from Hiron Point to Fairway Buoy does not have the required navigable depth for 9m draft vessel. Like any other tidal river, Pussur river maintains two distinct channels, flood and ebb channel as shown in Figure 1. If there is divergence of ebb and flood axes, this should be minimized as far as possible to get the full benefit of ebb and flood tidal energies without reducing tidal flux.

In the recent years from June 2000 to February 2010, MPA has carried out maintenance dredging of 3.0 million m^3 volume from Sabur Beacon to Southern Anchorage. However, within 3 months, due to high backfilling rate, jetty front has been filled up and also other areas like Sabur Beacon, Southern Anchorage channel near the confluence have also been filled up rapidly.

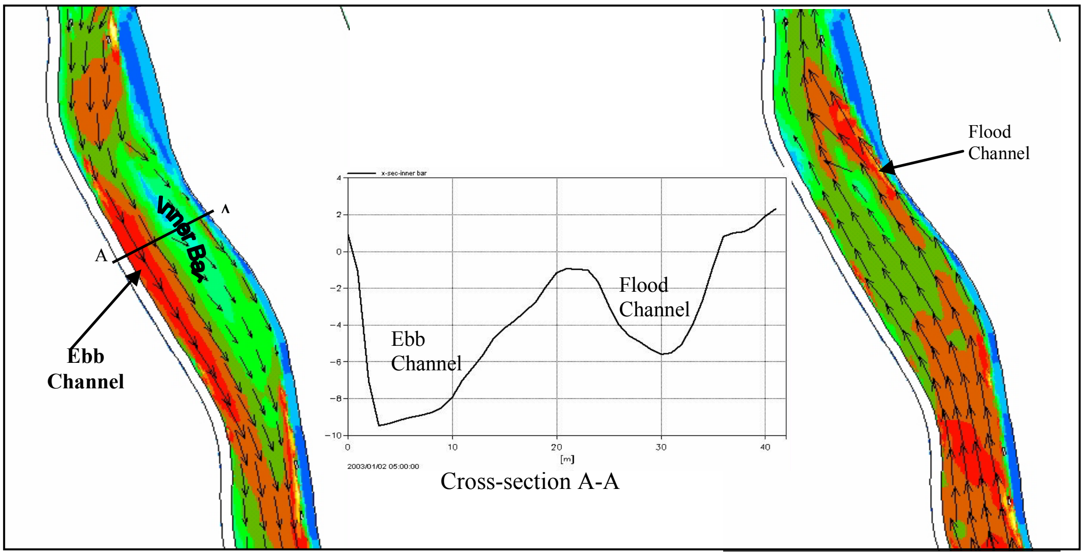

Figure 1: Divergence of Ebb and Flood channel near Inner bar; Left figure is during ebb tide and right figure is during flood tide. At middle, cross-section A-A on the inner bar started from west bank. Color contours are the speed; red color contour is for higher speed and green color for lower. Vectors are the direction of current.

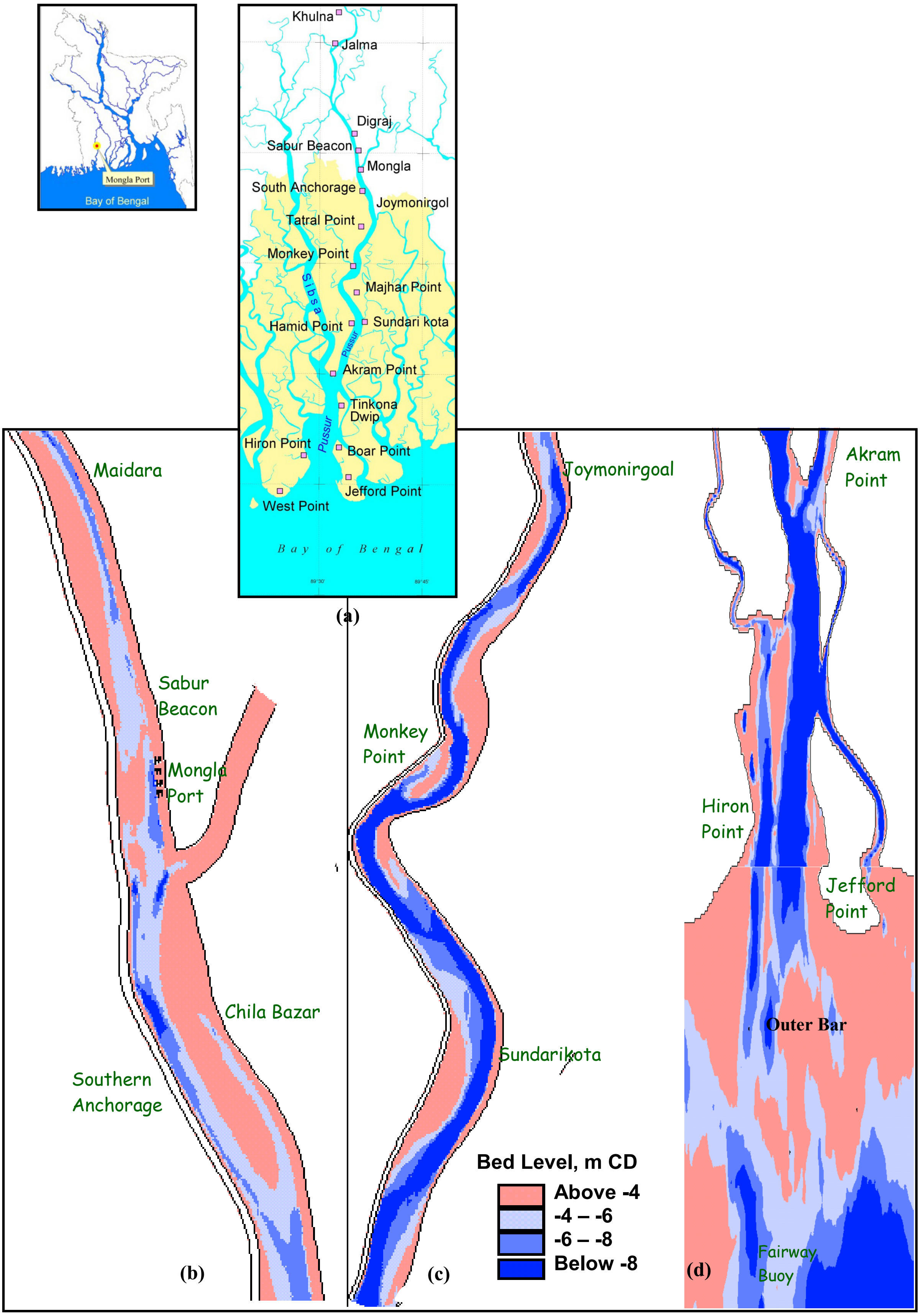

Figure 2: Present navigable depth in the Pussur river from Maidara to Fairway Buoy.(a) Location map, (b) river reach from Maidara to Joymonirgoal, (c)From Joymonirgoal to Zafar Khal, (d) Akram Point to Fairway Buoy; Deep to light blue is the bed level –8m to –4m CD @ 2 m.

4. OPTIONS FOR IMPROVEMENT OF NAVIGABILITY

The main principle of improvement is to maintain one uniform, deep channel by concentrating the tidal energy. To obtain full benefit of ebb and flood tide energies measures were considered to minimize the existing divergence of ebb and flood flows. In view of the existing physical processes of the Pussur river, the existing practice of dredging is the first option as short-term sustainability. However, dredging has an immediate benefit in such conditions, but tends to be prohibitive due to very short life. For long-term sustainable solution, several combinations of different engineering measures like river training works and tidal basins have been devised by using the calibrated modelling tools. Accordingly, the following three options have been studied for short and medium to long-term sustainable navigable channel.

Option-1: Dredging : Capital and Maintenance dredging (Figure 3 & 4)
Option-II: Dredging + Canalization through structural interventions (Figure 3 & 4)
Option-III: Dredging + Canalization through Structural Interventions + Tidal river Management (Figure 3 & 4)

Table 1 shows the different measures corresponding to various options for short, medium to long-term sustainable solution of the navigability problem.

Table 1: Options for Solution of Navigability Problem

Sl.	Structure/Intervention	Option-1	Option-2	Option-3
A	**Dredging**			
1	Capital dredging from Sabur Beacon to Southern Anchorage	√	√	√
2	Capital dredging near outer bar area	√	√	√
3	Maintenance Dredging of navigation channel at specific locations	√	√	√
B	**Canalization through Structural Interventions**			
4	Sheet Piling at jetty front		√	√
5	Guide Bundh opposite to jetty		√	√
6	Inner Bar Guide Bundh		√	√
7	Revetment at right bank opposite to inner bar		√	√
8	Revetment at the left bank upstream of jetty		√	√
9	Revetment at the right bank near confluence		√	√
C	**Catchment Management**			
0	Tidal Basin at left bank of Pussur river			√
1	Tidal basin at right bank of Mongla Nullah			√

4.1 Option 1- Dredging: Capital and maintenance dredging

The suggested solution in Option-I is the dredging for 7 m draft vessel to be utilized at the Mongla Port jetty. It is suggested that the deeper draft vessel of 9 m will reduce its load to 8 m draft at Akram point and will be anchored further upstream at downstream of Joymonirgoal and further lower draft vessel of 7 m will be utilized at the Mongla jetty. However, the study shows high backfilling rate from Sabur Beacon to Southern Anchorage channel and the dredged channel get silted up almost 100% within 5-6 months. To maintain the channel at design depth from Sabur Beacon to Southern Anchorage, 1.7 million m^3 maintenance dredging is required almost every year. On the other hand, the available and proposed spoil disposal area will be filled up within next 2-3 maintenance dredging programme. However, dredging as a solution of navigable problem is not a sustainable solution. If no engineering measures are taken, a sustainable navigable channel will not be possible to maintain. Also, in front of the jetty, frequent dredging is needed for the rapid siltation of the berthing pocket due to sliding of sediment from jetty underneath.

Outer Bar Improvement

At the approach channel in the outer bar area, for 9m draft vessel up to Akram Point, capital dredging (3.2 million m^3) followed by maintenance dredging (1.5 million m^3) has been suggested (Figure 4). However, in consideration of the magnitude of the natural changes, only dredging without structural intervention at the mouth of the Pussur river will not be sustainable and cost prohibitive. Frequent maintenance dredging will be necessary to maintain the navigable channel. Structural intervention as breakwater may maintain the navigation route with minimum dredging. However, this will involve exorbitant cost for building and maintaining more than 30 km of breakwaters.

4.2 Option-II: Canalisation and Dredging

The canalisation through structural intervention will converge the flood and ebb flow in the one channel and will thus increase the velocity (Figure 5). This increase of the velocity is in the range of 10-50% in the navigable channel at the constricted reaches. The increased velocity eventually will increase the depth of the channel by 1-2 m and river bed level will be maintained at –7m CD to –7.5m CD at the constricted reaches. Figure-6 is the cross-section at the southern anchorage shipping channel with and without guide bundhs. The result is after 2 months of implementation of guide bundh in the inner bar. Maintenance dredging will not be required at the Sabur Beacon, Jetty Front and Southern Anchorage channels. However, 0.4-0.6 million m^3 annual maintenance dredging is required at few locations, such as, confluence channel and downstream of inner bar. The maintenance dredging quantity may vary, this should be based on the available latest MPA hydrographic monitoring survey charts. To maintain the berthing pocket in front of the jetty, sheet piling is suggested to arrest the sliding of the sediment underneath jetty. However, the constriction may cause bank erosion at the upstream of the jetty and the western bank of inner bar, revetment is suggested at those locations.

Outer Bar Improvement

At the outer bar area approach channel up to Akram Point, capital dredging followed by maintenance dredging (as suggested in Option-I) has been suggested for this option also.

4.3 Option-III: Catchment Management and Canalization

The tidal river management in combination with canalization has been suggested in Option-III. The combined effects of these two will increase the velocity in the navigable channel from Sabur Beacon to downstream of inner bar. The construction of tidal basin will generate 10% increase of tidal volume at and around jetty area. There will be 4% increase of tidal volume near inner bar in the Pussur river. Due to this increased tidal volume, the flow velocity in the navigable channel will be increased about 10 to 15% in comparison to Option-II. Eventually, the navigable channel will be deepened and the combined effect will maintain a sustainable navigable channel at –7m CD to –7.5m CD from Sabur Beacon to downstream of Inner Bar. During the operation of tidal basin, sedimentation would take place into the tidal basin that is suggested to dredge every 5 years to keep the tidal basin alive to flush the incoming sediment into the river during high tide. This option has considerable merit as it would contribute to increase tidal flow in the Pussur river and it is also important to restore part of the tidal volume lost in former years in these rivers.

Outer Bar Improvement

At the outer bar area approach channel up to Akram Point, capital dredging followed by maintenance dredging (as suggested in Option-I) has been suggested. However, frequent maintenance dredging will be necessary.

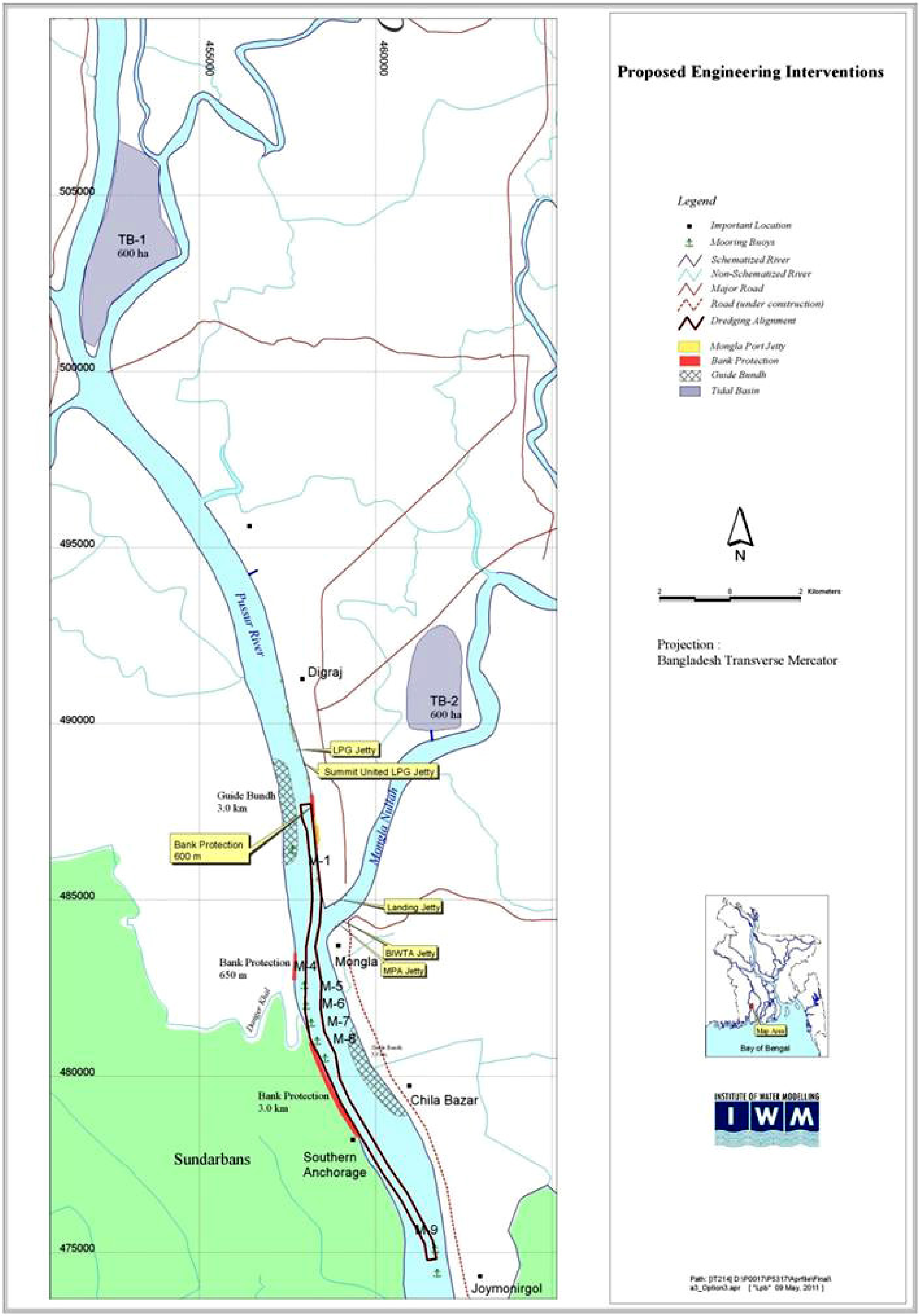

Figure 3: Options for improvement of navigability.

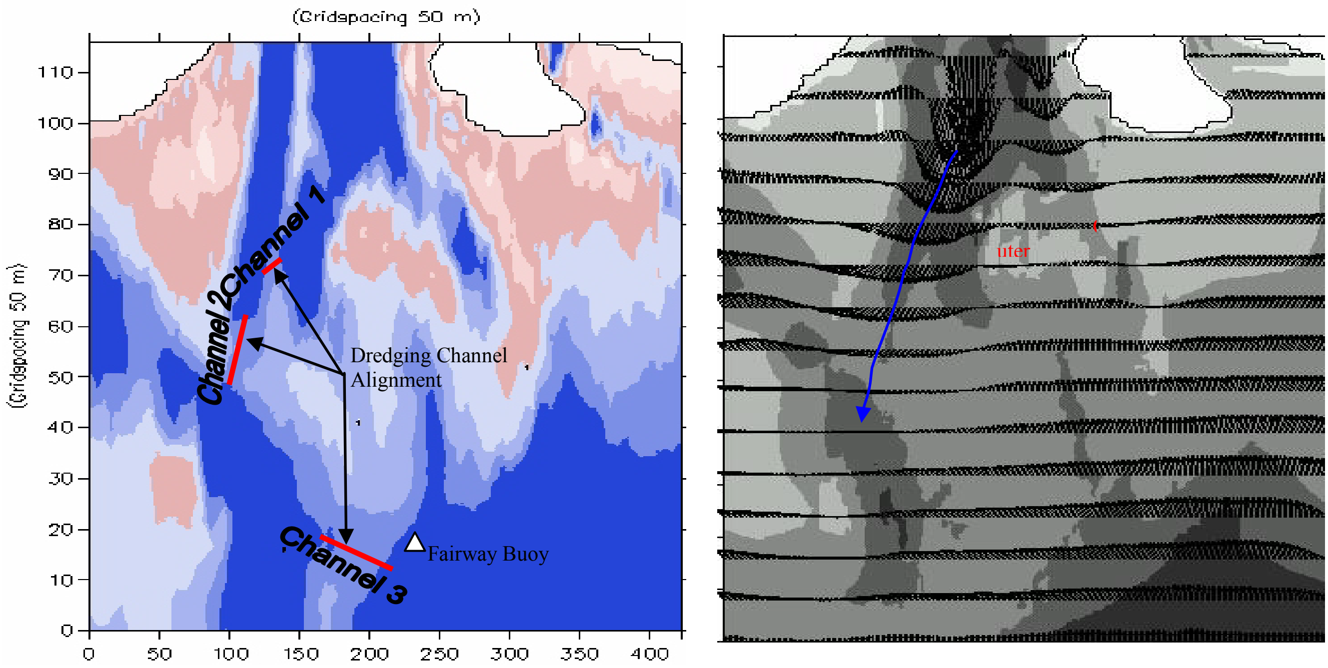

Figure 4: Dredging alignment for Outer Bar improvement

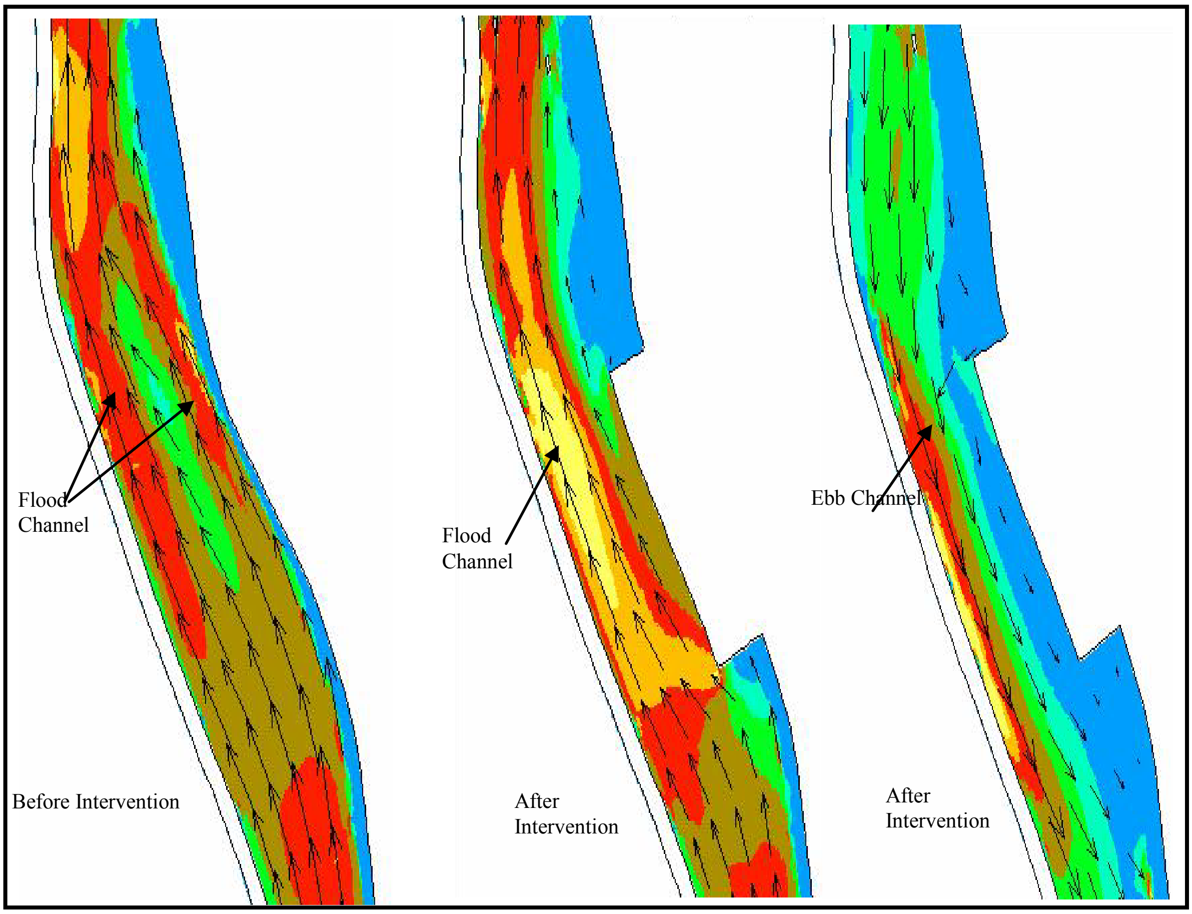

Figure 5: Convergence of Flood channel after constriction along the western channel near Inner Bar. Arrows are the direction of flood and ebb tide. Color legend for speed: Red to yellow is higher speed, green to blue is lower speed.

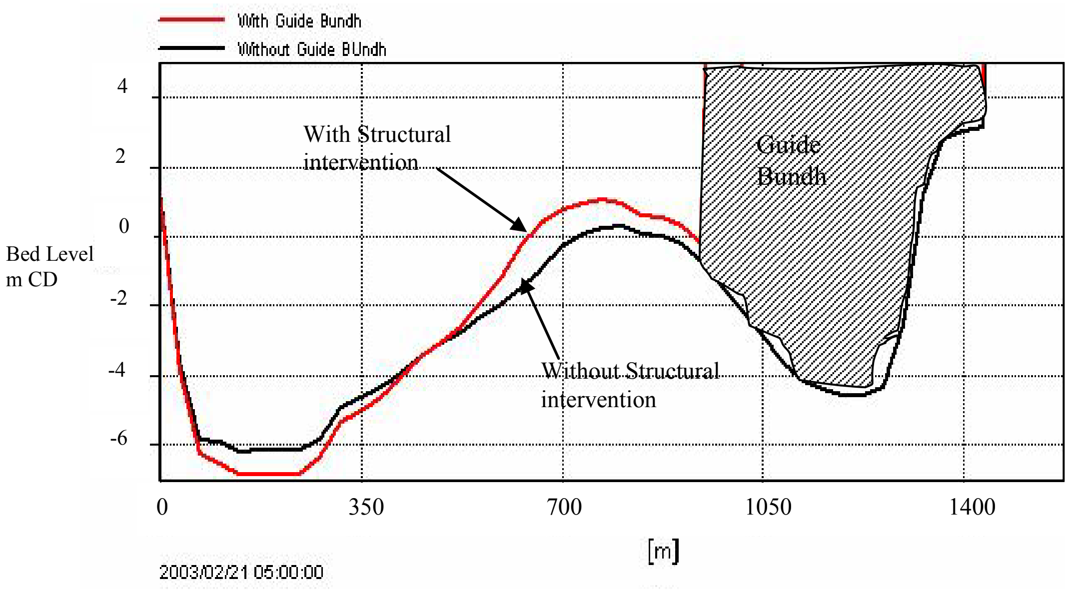

Figure 6: Improvement of navigable channel due to canalisation. The bed level of a cross-section at southern anchorage shipping channel, distance from western bank. The bed levels are in Chart Datum.

5. DESIGN OF ENGINEERING INTERVENTIONS

Considerable efforts have been given to make feasibility level design for all potential engineering interventions with respect to improvement of navigability with the aim to assess the project investment cost as well as to make financial and economic analysis of the project. The feasibility level design has been performed based on the design criteria contained in Bangladesh Water Development Board (BWDB) publication "STANDARD DESIGN MANUAL" VOLUME 1. Findings of the present and previous study results and most recent hydrometric and bathymetric survey data were taken into account in selecting the required design data.

6. ENVIRONMENTAL AND SOCIAL IMPACT ASSESSMENT

The EIA has been carried to identify the likely impacts, both positive and negative, of the proposed interventions, and then to quantify and, where possible, value these so that they can be used in a multi-criteria analysis for rational decision making. The overall aim is to ensure that any approved intervention is environmentally sound and sustainable. The EIA component of the proposed Study follows the WARPO guidelines for EIA (WARPO, November 2001), The WARPO EIA Guidelines follow the spirit of World Bank Operational and OECD Directives 4:01 on Environmental Assessment and 4:30 for Involuntary Resettlement. The scope of the EIA covers the natural and human environment, their interaction and any induced change brought about by the proposed interventions. The methodology compares the present situation to that in the future both with and without the proposed intervention. The major components of the Study are:

- Social impact assessment due to proposed intervention (s);
- Public Participation (PP) program to gain local people's reaction to the proposed intervention and modify it to satisfy their interests provided they are technically and economically feasible, and
- An Environmental Impact Assessment (EIA) using the WARPO EIA Guidelines.

A precise overview of the assessment of the environmental impacts from the different project elements is presented in Table 2. For this assessment, only the main impacts are presented considering the following parameters: (a) Effects on navigability, (b) Land acquisition, (c) Ecology and (d) Socio-economic conditions.

Table 2: Evaluation of impacts of different project elements

Ref. Nr.	Structure / Intervention	Navigability	Acquisition of land	Ecology	Socio-Economic Development
1	River training at jetty	+	0	0	+
2	River Training at southern anchorage (inner bar)	+	0	0	+
3	Construction of tidal Basins along the Pussur and Mongla Nulla	++	--	0/-	++
4	Dredging along jetty and across confluence	+	-	-	+
5	Dredging along Southern anchorage	+	0	-	+
6	Dredging and river training	++	0	-	++

--	Severe negative impact	+	some benefit
-	negative impact	++	major benefits
0	no or insignificant impact		

Due to the nature of the proposed engineering interventions negative impacts on the different components of the environment may occur. It is expected that such effects will mostly be limited to the direct vicinity of the project works and the river water on the both upstream and downstream of the works. The construction activities for river training works excluding the option of dredging only are deemed to produce relatively minor environmental impacts. The environmental impact assessment confirms that potential adverse impacts are minimal and can be mitigated through proper measures.

7. ECONOMIC VIABILITY

7.1 Project Investment Cost

The cost estimate is prepared for all engineering interventions on the basis of preliminary drawings. These cost are used for economic and financial analysis of the project. Before execution of the works detail construction drawing and cost estimate shall have to prepared on the basis of detail survey and design. The unit rates used in the cost estimate have been taken mostly from Standard Schedule of Rates of Khulna O & M Circle of Bangladesh Water Development Board. Other rates are assumed considering the present market rate. The summary of project investment cost is furnished in the Table 3.

Table 3: Summary of Project Investment Cost

Sl. No.	Project Component	Quantity	Cost (Lakh Tk.)
1	Guide bundh at inner bar	3000m	9266
2	Guide bundh at opposite to Jetty	3000m	8997
3	Bank revetment at opposite to inner bar guide bundh	3000m	6815
4	Bank revetment at Jetty	600m	1363
5	Bank revetment at left bank of Pussur river below the confluence of Pussur river and Mongla Nala	650m	1477
6	Sheet pile wall	1015m	2902
7	Tidal basin , TB1	400 ha	6546
8	Tidal basin , TB2	400 ha	6546
9	Capital dredging	4.90mm3	8610
	Total		**52522**

The project justification was evaluated by conducting the financial and economic analysis as well as considering other potential effects of different parameter upon various indicators. The project impacts on economic welfare of the project on the economy as a whole were assessed by project evaluation along with possible different alternatives. On the basis of above assumptions and procedures economic and financial analyses were carried out and the results are summarized in Table 4.

Table 4: Analytical Results

Viability Indicator	Option-1		Option-2		Option-3	
	Financial	Economic	Financial	Economic	Financial	Economic
Capital cost (Lakh Taka)	8610.00	7766.22	39430	32552.88	52522	37328.88
Annual maintenance cost (Lakh Taka)	6010.00	5421.02	7405.20	6406.70	6310.40	5419.19
BCR @ 12%	0.94	0.89	0.98	1.00	1.19	1.35
NPV (Lakh Taka) @ 12%	(-) 3074.67	(-) 4890.35	(-) 1640.77	(-) 190.46	(+) 13062.75	(+) 18640.54
IRR (%)	9.81	8.47	11.37	12.20	16.96	22.31

Based on the above results, Option-III is found to be economically viable. Sensitivity analysis were carried out on the recommended option based on engineering exercise in order to test the effects of changes and the results are presented in Table 5.

Table 5: Sensitivity Analysis

Critical Variable Viability Indicators	Option-3 (Recommended Option)
	EIRR (%)
Base case	22.31
Investment cost and maintenance cost increased by 20%	15.46
Costs increased by 10% and benefits reduced by 10%	15.09
Benefits reduced by 20%	14.60
Implementation period delayed by 2 years	17.87

8. MULTI-CRITERIA ANALYSIS OF DIFFERENT OPTIONS

The multi-criteria analysis has been carried out to evaluate the different options. Table 6 presents the results of the analysis. It is expected with this type of analysis, that the units of measurement cannot be same for each criterion, however the relative importance can be assessed and can be used in selecting the best option. By comparing the different options considering the criteria given in the table, it is evident that the constriction with the guide bundhs, revetment and tidal basin i.e. Option III is more feasible. However, implementation of any option requires physical and environmental monitoring and evaluation for proper operation and maintenance of navigation route.

Table 6: Multi-Criteria Analysis of different Options

		Navigation Improvement of Mongla Port		
		Option 1	Option 2	Option 3
Economic Criteria	EIRR	8.47%	12.20%	22.31%
Navigation Route Management Criteria	Better project Implementation	5+	4+	4+
	O & M Cost Recovery	3+	4+	5+
	Post Construction O & M	3+	4+	4+
Environmental Criteria	Terrestrial Environment	2+	3+	3+
	Aquatic Environment	2+	3+	4+
	Navigation	2+	4+	5+
	Nutrition	2+	2+	2+
	Bio-diversity	2+	3+	3+
	Water Pollution	-2	-2	-2
Quantitative Analysis	Employment for Construction (lakh man-day)	149	225	289
	Annual Employment for O & M (lakh man-day	3	12	15
Social Criteria	Linkage Employment	2+	4+	4+
	Social acceptance	5+	4+	3+
	Community Stability	3+	3+	3+
	Quality of life	4+	4+	5+
	Land Acquisition	Nil	Nil	Yes
	Communication	3+	4+	4+
	Income Distribution	2+	4+	4+
Industrial Criteria	Industrial Development	1+	3+	4+

Note : The impact assessment scoring was done within the following points score scale ranking from –1 to –5 for negative impact and from +1 to +5 for positive impact.

9. PROJECT IMPLEMENTATION PHASE

The project will be implemented in three phases as presented in Table 7.

Table 7: Implementation Schedule

Phase	Project components	Start year	End year
1	1. Capital dredging 2. Sheet pile wall at Jetty 3. Guide Bundh at opposite to Jetty 4. Bank revetment at Jetty	1st Year 1st Year 2nd Year 2nd year	1st Year 2nd Year 3rd Year 3rd Year
2	1. Guide bundh at inner bar 2. Bank revetment opposite to inner bar	3rd Year 3rd Year	4th. Year 4th. Year
3	1. Tidal Basin 2. Bank revetment opposite to confluence	4th Year 4th Year	5th Year 5th Year

10. CONCLUSION

The suggested solution in Option-I is the dredging for 7m draft vessel to be utilized at the Mongla Port jetty. It is suggested that the deeper draft vessel of 9m will reduce its load to 8m draft at Akram point and will be anchored further upstream at downstream of Joymonirgoal and further lower draft vessel of 7m will be utilized at the Mongla Jetty. The canalization through structural intervention increases the velocity in the range of 10-50% in the navigable channel. The increased velocity eventually increase the depth of the channel by 1-2m and river bed level will be maintained at –7m CD to –7.5m CD at the constricted reaches. The tidal river management in combination with canalization has been suggested in Option-III. The combined effects of these two will increase the velocity in the navigable channel from Sabur Beacon to downstream of inner bar.

11. RECOMMENDATION

i. The Option-III has been found as a technically feasible, environment friendly, economically viable and socially acceptable solution. This option is recommended for implementation as a long-term solution for maintaining a sustainable navigable channel from Sabur Beacon to Fairway Buoy.

ii. Before implementation of the suggested solution, the design needs to be reviewed to the changed hydrodynamic and morphological conditions.

iii. Monitoring and evaluation is required to observe the performance, comparing it with the targets and identifying shortfalls. Monitoring will generate timely information, which subsequently will enable the decision makers to take corrective measures for maintaining a proper navigation channel.

iv. A good navigable channel is maintained from Joymonirgoal to Akram Point over the last two decades. The communication from the present port jetty to Joymonirgoal will be improved after the construction of the proposed connecting road. In that consideration, Joymonirgoal may be considered as a suitable location for the future extension of the present port.

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REFERENCES

DHI (1990), Mathematical Model Study of Pussur-Sibsa River System and Karnafuli River Entrance, Final Report, Volume l

DHI (1993), Mathematical Model Study of Pussur-Sibsa River System and Karnafuli River Entrance, Final Report, Annex 1 & 2
Farleigh D.R.P. (1981), Pussur River Study Phase I, Final Report, Port of Chalna Authority, February 1981
Farleigh D.R.P. (1984), Pussur River Study Phase II, Final Report, Port of Chalna Authority, February 1984
IWM (2002), Updating and Validation of General Model and Six Regional Models
Japan Overseas Consultants Co., Ltd, Frederic R. Harris, Inc, Bangladesh Consultants Ltd (1996), Mongla Port Area Development Project, Asian Development Bank, Appendices to Final Report, Volume I.
Japan Overseas Consultants Co., Ltd, Frederic R. Harris, Inc, Bangladesh Consultants Ltd, 1996, Mongla Port Area Development Project, Asian Development Bank, Appendices to Final Report, Volume II.
Mott MacDonald, Ports Upgrading Project (May-1998), Asian Development Bank , Government of the People's Republic of Bangladesh,Final Report.
Shahjahan Shah Md, Department of Hydrography, BIWTA, Dhaka (1983), Pussur-Sibsa River System in Bangladesh: Report on Tidal Discharge and level Data Collected During phase I & II Study for the Port of Chalna Authority, 1980-82.
Viability and the Future of Chalna Port (1980), Port of Chalna Authority, Khulna.

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THE LIVELIHOOD VULNERABILITY INDICES TO ASSESS THE CLIMATE CHANGE INDUCED RISKS -A CASE STUDY OF COASTAL BANGLADESH

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ABSTRACT

The South-West coastal region of Bangladesh is extremely vulnerable to natural and climate change-related disasters. The livelihood vulnerability index (LVI) were calculated based on two methods named as Composite LVI and LVI-IPCC to estimate climate change vulnerability in four coastal villages named Debhata, Morreganj bazar, Koira-3 and Labonchora. About 100 households were surveyed in each place to collect data on socio-demographics, livelihoods, social networks, health, food and water security, natural disasters and climate variability parameters. It is found that the composite approach LVI calculation is suitable for community or district level whereas the LVI-IPCC is suitable for regional level evaluation. The LVI index shows that Koira is more vulnerable in terms of natural disaster and health profile, while Debhata is more vulnerable in terms socio-demographic, livelihood vulnerability and food profile, Morrelganj is more vulnerable in terms of water resources and social network profile.

Keywords: *Vulnerability Index, Climate Change, IPCC, Coastal Area*

1. INTRODUCTION

Climate change is now the major global concern. The climate is changing and weather patterns are becoming extreme and unpredictable (IPCC, 2007). Disruptions of the earth's atmosphere in terms of rise in temperature due to increase in the amount of greenhouse gases have resulted in increased frequency and intensity of extreme weather events like cyclones, floods, heavy rainfall and droughts. Bangladesh is a tropical country and extremely vulnerable to various natural disasters such as floods, cyclones, tornadoes, tidal surges, storm surges, river bank and coastal erosion, and droughts. It is currently ranked as the most climatically vulnerable country in the world (EC, 2008). There are approximately 711 km of costal area in Bangladesh (CDP, 2006). Over a period of 100 years, 508 cyclones have affected the Bay of Bengal Region of which 17 per cent caused serious land erosion (GoB, 2008).The majority of the population have experienced disasters in varying degrees and forms. It has affected Bangladesh by land erosion, salinity intrusion and loss in biodiversity. Sea level rise has different impacts on Bangladesh. A one-meter sea level rise (SLR) will affect the country's vast costal area and flood plain zone. Most vulnerable sectors to one meter sea level rise are costal resources, water resources, agriculture and the ecosystem of Bangladesh.

In the month of April-May and September-November is known as 'Cyclone season', these natural and climate-related disasters have a significant and lasting impact on their lives and livelihoods of extreme poor, particularly in 12 districts including Khulna and Bagerhat. Cyclone Aila, a tidal surge, struck the South-West coast on the 25th of May 2009 with 13 ft. high water, which causes breaking of river embankments and dykes in several places, washing away the lives and livelihoods of people in the South-West costal belt. Before Aila, cyclone Sidr hit the coastal areas of Bangladesh on November 15, 2007.

Adaption and mitigation are two best and only choice for Bangladesh. The first one is country specific or local specific where the mitigation demands collective efforts if global communities. These affected communities struggle to adapt to climate change. Globally, climate policies of developed nations including increased reliance on biofuels may have a detrimental impact on staple food markets and consequently the nutrient needs of already malnourished populations (Boddiger, 2007). However, people in the coastal area vulnerable to cyclone, tidal surges and river erosion, along with salinized water and soil. Among all, the extreme poor people are suffering the most because of their deep dependence on nature for their lives and livelihood. They live in physically fragile huts that offer little protection from floods, storms and cyclones. They do not have their own homestead land and are often forced to live on marginal and flood-prone land. Vary often they lose their homes due to river erosion but are often too poor to buy land elsewhere as a coping mechanism (TEARFUND, 2005).

1.1 Climate Change Vulnerability Assessment

Vulnerability assessment describes a diverse set of methods used systematically integrate and examine interactions between humans and their physical and social surroundings. Vulnerability assessments have been used in a variety of contexts including the USAID Famine Early Warning System (FEWS-NET) (USAID, 2007a), the World Food Program's Vulnerability Analysis and Mapping tool for targeting food aid (World Food Program, 2007), and a variety of geographic analysis combining data on poverty, health status, biodiversity, and globalization. The field of climate vulnerability assessment has emerged to address the need to quantify how communities will adapt to changing environmental conditions. Various researchers have tried to bridge the gap between the social, natural, and physical sciences and contributed new methodologies that confront this challenge (Polsky et al., 2007). Many of these rely heavily on the IPCC working definition of vulnerability as a function of exposure, sensitivity, and adaptive capacity (IPCC, 2001). Exposure in this case is the magnitude and duration of the climate-related exposure such as a drought or change in precipitation, sensitivity is the degree to which the system is affected by the exposure, and adaptive capacity is the system's ability to withstand or recover from the exposure (Ebi et al., 2006). Fusel and Klein (2006) divide available studies into first-generation vulnerability assessment based on climate impact assessments relative to baseline conditions, and second-generation assessments that incorporate adaptive capacity. Of the second-generation studies, there are a multitude of interpretation about how best to apply exposure, sensitivity, and adaptive capacity concepts to quantify vulnerability (Sullivan, 2002; Ebi et al., 2006; Thornton et al., 2006; Polsky et al., 2007).

1.2 The Livelihood Vulnerability Index

The Sustainable Livelihood Approach, which looks at five types of household assets-natural, social, financial, physical and human capital (Chambers and Conway, 1992), is an approach used to design development programing at the community level (United Nations General Assembly, 1997). The approach has proven useful for assessing the ability of households to withstand shocks such as epidemics or civil conflict. Climate change adds complexity to household livelihood security. The Sustainable Livelihood Approach to a limited extend address this issue of sensitivity and adaptive capacity to climate change , but a new approach for vulnerability assessment that integrates climate exposures and accounts for household adaptation practices is needed in order to comprehensively evaluate livelihood risks resulting from climate change.

The Livelihood Vulnerability Index (LVI) uses multiple indicators to assess exposure to natural disasters and climate variability, social and economic characteristics of households that affect their adaptive capacity and current health, food and water resource characteristics that determine the sensitivity to climate change impacts. Two approaches are presents: the first expresses the LVI as a composite index build of seven major components while the second particles the seven into IPCC's three contributing factors to vulnerability-exposure, sensitivity and adaptive capacity.

The LVI approach focused on quantifying the strength of current livelihood and health systems as well as the capacity of communities to alter these strategies in response to climate-related exposures. The LVI is designs to provide development organizations, policymakers and public health practitioners with a practical tool to understand demographic, social and health factors contributing to climate vulnerability at the district or community level. It is designs to flexible so that development planners can refine and focus their analyses to suit the needs of each geographic area. In addition to the overall composite index, sectorial vulnerability scores can be segregated to identify potential areas for intervention. This study is carried out to evaluate the most vulnerable livelihood parameters based on two methods: developed by Sullivan et al. (2002) named as composite LVI and by IPCC (2001) named as LVI-IPCC.

2. Methodology

2.1 Composite Index Approach

The LVI includes seven major components: Socio-demographic Profile, Livelihood Strategies, Social Networks, Health, Food, Water and Natural Disasters and Climate Variability. Each is consist of several indicators or sub-components (Table-1). These were developed bases on a review of the literature on each major component, for example studies on satkhira's water sector, as well as the practicality of collecting the needed data through household surveys. Table includes an explanation of how each sub-component was quantified, the survey question used to collect the data, the original source of the survey question, and potential sources of bias.

The LVI uses balance weighted average approach (Sullivan et al., 2002) where each sub-component contributes equally to the overall index even through each major component is comprised of a different number of sub-components. The LVI formula uses the simple approach of applying equal weights to all major components. This weighting scheme could be adjusted by future users as needed.

Because each of the sub-components is measured on a different scale, it was first necessary to standardize each as an index. The equation is used for this conversion was adapted from that used in the human development index to calculate the life expectancy index, which is the ratio of the difference of the actual life expectancy and a pre-selected mini um and the range of pre-determined maximum life expectancy (UNDP, 2007):

$$index_{s_d} = \frac{S_d - S_{min}}{S_{max} - S_{min}} \tag{1}$$

Where S_d is the original sub-component for district d, and S_{min} and S_{max} are the minimum and maximum values, respectively, for each sub-component determined using data from both districts. For example, the 'the average time travel to primary water source' sub-component ranged from 1 to 160 min in the four districts i surveyed. These minimum and maximum values were used to transform this indicator in to a standardized index so it could be integrated into the water component of the LVI. Some sub-components such as the 'average agricultural livelihood diversity index' were created because an increase in the crud indicator, in this case the number of livelihood activities undertaken by a household, was assumed to decrease vulnerability. The maximum and minimum values were also transformed following this logic and eq. (1) used to standardize these sub-components.
After each was standardized, the sub-components were averaged using Eq. (2) to calculate the value of each major component:

$$M_d = \frac{\sum_{i=1}^{n} index_{s_{di}}}{n} \tag{2}$$

Where M_d=one of the seven major components for district *d* [Socio-Demographic Profile (SDP), Livelihood Strategies (LS), Social Network (SN), health (H), Food (F), Water(W), or Natural disasters and Climate Variability (NDCV)], $index_{s_{di}}$ represents the sub-components, indexed by i that make up each major component , and n is the number of sub-components in each major component. Once values for each of the seven major components for a district were calculated, they were averaged using Eq. (3) to obtain the district-level LVI:

$$LVI_d = \frac{\sum_{i-1}^{7} w_{M_i} M_{di}}{\sum_{i-1}^{7} w_{M_i}} \tag{3}$$

This can also be expressed as

$$LVI_d = \frac{w_{SDP} SDP_d + w_{LS} LS_d + w_{SN} SN_d + w_H H_d + w_F F_d + w_W W_d + w_{NDC} NDCV_d}{w_{SDP} + w_{LS} + w_{SN} + w_H + w_F + w_W + w_{NDCV}} \tag{4}$$

Where LVI_d is the Livelihood Vulnerability Index for district *d* equals the weighted average of the seven major components. The weights of each major component, w_{M_i},are determine by the number of sub-components that make up each major component and are included to ensure that all sub-components contribute equally to the overall LVI (Sullivan et al., 2002) in this study, the LVI is scaled from 0 (least vulnerable) to 0.5 (most vulnerable). For informative purposes, a detailed example of calculating the Food major component for LVI is presented in Appendix A.

2.2 IPCC Framework Approach

Here developed an alternative method for calculation the LVI that incorporates the IPCC vulnerability definition. Table 2 shows the organization of seven major components in the LVI (IPCC) framework. Exposure of the stud y population is measured by the number of natural disasters that occurred in the past 6 years, while climate variability is measured by the average standard deviation of the maximum and minimum monthly precipitation over 6- year period. Adaptive capacity is quantified by the demographic profile of a district (e.g., percent of female-headed households), the type of livelihood strategies employed (e.g. predominately agricultural, or also collect natural resources to sell in the market), and the strength of social networks (e.g. percent of residents assisting neighbors with chores). Last sensitivity is measures by assessing the current state of a district's food and water security and health status.

The same sub-components outlined in Table 1 as well as Equations (1) - (3) were used for calculate the LVI (IPCC). The LVI (IPCC) diverges from the LVI when the major components are combined. They are first combined by following Equation (5):

$$CF_d = \frac{\sum_{i-1}^{n} w_{M_i} M_{di}}{\sum_{i-1}^{n} w_{M_i}} \quad (5)$$

Table 9: Major components and sub-components comprising the Livelihood Vulnerability Index (LVI) developed for Debhata, Morrelganj, Koira and Labonchora, Bangladesh.

Major components	Sub-component	Explanation of sub-components	Survey question	Source
Socio-demographic profile	Dependency ratio	Ratio of the population under 15 and over65 years of age to the population between 19 and 64 years of age.	Could you please list the ages and sexes of every person who eats and sleeps in this house? If you had a visitor who ate and slept here for the last 3 days.	Adapted from Domestic Household Survey (DHS) (2006). Measure DHS: Model Questionnaire with commentary
	Percent of female-headed households	Percentage of house- holds where the Primary adult is female. If a male head is away from the home >6 months per year the female is counted as the head of the household.	Are you the head of the household?	Adapted from DHS (2006)
	Percent of literacy	Percent of literacy of the district	Percent of literacy in this district?	Developed for the purposes of this questionnaire.
	Percent of house-holds where head of house hold has not attended school	Percentage of house -holds where the head of the household reports that they don't attended school.	Did you ever go to school?	Adapted from DHS (2006)
	Percent of households with orphans	Percentage of house- holds that have at least 1 orphan living in their home. Orphans are children <18 years old who have lost one or both parents.	Are there any children less than 18 years old from other families living in your house because one or both of their parents has died?	Adapted from DHS (2006)
Livelihood	Percent of households with family member working in a different community	Percentage of households that report at least 1 family member who works outside of the community for their primary work activity.	How many people in your family go to a different community to work?	Adapted from World Bank (1997). Household Questionnaire: Survey of Living Conditions , Uttar Pradesh and Bihar
	Percent of households dependent solely on agriculture as a source of income	Percentage of households that report only agriculture as a source of income.	Do you or someone else in your household raise animals? Do you or someone else in your household grow crops? Do you or someone else in your household collect something from the bush, forest or rivers to sell?	Adapted from World Bank (1997)
Health	Average time to health facility	Average time it takes the house holds to get to the	How long does it take you to get to a health facility?	Adapted from World Bank (1997)

	(minutes)	nearest health facility.		
	Percent of households with family member with chronic illness	Percentage of house- holds that report at least 1 family member with chronic ill- ness. Chronic ill- ness was defined subjectively by respondent	Is anybody in your family chronically ill (they get sick very often)?	Adapted from DHS (2006)
Major components	**Sub-component**	**Explanation of sub-components**	**Survey question**	**Source**
	Average Malaria Exposure*Prevention Index (range: 0–12) a	Months reported exposure to Malaria*Owning at least one bed net indicator (have bed net = 0.5, no bed net = 1) (e.g., Respondent reported malaria is a problem in January –March and they do not own a bed net= 3*1 = 3).	Which months of the year is malaria particularly bad? How many mosquito nets do you have?	Malaria: Adapted from WHO/ RBM (2003). Bed nets: DHS (2006)
Social Networks	Average Receive: Give ratio (range: 0–15)	Ratio of (the number of types of help received by a household in the past month + 1) to (the number of types of help given by a household to someone else in the past month + 1)	In the past month, did relatives or friends help you and your family: (e.g., Get medical care or medicines, Sell animal products or other goods produced by family, Take care of children) In the past month, did you and your family help relatives or friends: (same choices as above)	Adapted from World Bank (1997)
	Average Borrow: Lend Money ratio (range: 0.5–2)	Ratio of a household borrowing money in the past month to a household lending money in the past month, e.g., If a household borrowed money but did not lend money, the ratio = 2:1 or 2 and if they lent money but did not borrow any, the ratio = 1:2 or 0.5.	Did you borrow any money from relatives or friends in the past month? Did you lend any money to relatives or friends in the past month?	Adapted from DHS (2006)
	Percent of households that have not gone to their local government for assistance in the past 12 months	Percentage of households that reported that they have not asked their local government for any assistance in the past12 months.	In the past 12 months, have you or someone in your family gone to your community leader for help?	Adapted from WHO/RBM (2003)
Food	Percent of households dependent on family farm for food	Percentage of households that get their food primarily from their personal farms.	Where does your family get most of its food?	Hahn *et al*., 2009
	Average number of months households struggle to find food (range: 0–12)	Average number of months households struggle to obtain food for their family	Does your family have adequate food the whole year, or are there times during the year that your family does not have enough food? How many months a year does your family have trouble getting enough food?	Adapted from DHS (2006)
	Average Crop Diversity Index (range: >0–1)a	The inverse of (the number of crops grown by a household +1). e.g., A house hold that grows	What kind of crops does your household grow?	Adapted from DHS (2006)

		pumpkin, maize, beans, and cassava will have a Crop Diversity Index = 1/ (4 + 1) = 20.		
	Percent of households that do not save crops	Percentage of households that do not save crops from each harvest.	Does your family save some of the crops you harvest to eat during a different time of year?	Hahn *et al*., 2009
	Percent of households that do not save seeds	Percentage of households that do not have seeds from year to year.	Does your family save seeds to grow the next year?	Hahn *et al*., 2009
Major components	**Sub-component**	**Explanation of sub-components**	**Survey question**	**Source**
Water	Percent of households that utilize a natural water source	Percentage of households that report a creek, river, lake, pool, or hole as their primary water source.	Where do you collect your water from?	Adapted from DHS (2006)
	Percent of households that do not have a consistent water supply	Percentage of households that report that water is not available at their primary water source everyday	Is this water available every day?	Adapted from DHS (2006)
	Inverse of the average number of liters of water stored per household (range:>0–1)	The inverse of (the average number of liters of water stored by each household + 1).	What containers do you usually store water in? How many? How many liters are they?	Hahn *et al*., 2009
Natural disasters and climate variability	Average number of flood, drought, and cyclone events in the past 6 years (range:0–7)	Total number of floods, droughts, and cyclones that were reported by households in the past 6 years.	How many times has this area been affected by a flood/cyclone /drought in 2006-2012	Adapted from Williamsburg Emergency Mgmt. (2004) Household Natural Hazards Preparedness Questionnaire.
	Percent of households that did not receive a warning about the pending natural disasters	Percentage of households that did not receive a warning about the most severe flood, drought, and cyclone event in the past 6 years.	Did you receive a warning about the flood/cyclone/ drought before it happened?	Adapted from Williamsburg Emergency Management (2004)
	Percent of households with an injury or death as a result of the most severe natural disaster in the past 6 years	Percentage of house -holds that reported either an injury to or death of one of their family members as a result of the most severe flood, drought, or cyclone in the past 6 years	Was anyone in your family injured in the flood/cyclone drought? Did anyone in your family die during the flood/ cyclone/drought?	Hahn *et al*., 2009
	Mean standard deviation of the daily average maximum temperature by month	Standard deviation of the average daily maximum temperature by month between 1998 and 2003 was averaged for each district	2001–2008: provincial data; weather station based in the provincial capital	Instituto Nacional de Estatistica (2007)
	Mean standard deviation of the daily average minimum temperature by month	Standard deviation of the average daily minimum temperature by month between 1998 and 2003 was averaged for each province.	2001–2008: provincial data; weather station based in the provincial capital	Instituto Nacional de Estatistica (2007)
	Mean standard deviation of average	Standard deviation of the average monthly Preci- pitation between 1998 and	2001-2008: provincial data; weather station based in the	Instituto Nacional de Estatistica (2007)

	precipitation by month	2003 was averaged for each province	provincial capital	

a) Some indicators such as the Livelihood Diversity Index were created because an increase in the crude indicator, in this case, the number of livelihood activities undertaken by a household, decreases vulnerability (e.g., a household who farms and raises animals is less vulnerable than a household who only farms) so by taking the inverse of the crude indicator, we create a number that reflects this line of reasoning and assigns higher values to households with a lower number of livelihood activities.

Table 10: Categorization of major components into contributing factors from the IPCC's vulnerability definition for calculation of the LVI–IPCC.

IPCC contributing factors to vulnerability	Major components
Exposure	Natural disasters and climate variability
Adaptive capacity	Socio-demographic profile
	Livelihood strategies
	Social networks
Sensitivity	Health
	Food
	Water

Where CF_d is an IPCC-defines contributing factor (exposure, sensitivity, or adaptive capacity) for district *d*, M_{di} are the major components for district *d* indexed by *i*, w_{M_i} is the weight of each major components in each contributing factor. Once exposure, sensitivity, and adaptive capacity were calculated, the three contributing factors were combined using the following equation:

$$LVI(IPCC_d) = (e_d - a_d) * s_d \quad (6)$$

Where $LVI(IPCC_d)$ the LVI for district d is expressed using the IPCC vulnerability framework. *e* is the calculated exposure score, *a* is the calculated adaptive capacity score and *s* is the calculated sensitivity score for district *d*. here scales the LVI(IPCC) from -1 (least vulnerable) to 1 (most vulnerable). For informative purposes, a detailed example of calculating the contributing factor of the LVI (IPCC) is presented in Appendix B.

2.3 Study Area

The study was undertaken for the tested of the LVI and LVI–IPCC in the Debhata, Morrelganj, Labonchora and koira village of Bangladesh during 2013. These were selected by rapid disaster impact map: based on analysis of MODIS satellite imagery, and the climate change issues confronting each. According to the coastal zone policy (CDP, 2005) of the Government of Bangladesh, 19 districts out of 64 are in the coastal zone covering a total of 147 upazillas of the country. Out of these 19 districts, only 12 districts meet the sea or lower estuary directly. Table 3 shows the percent of households were surveyed with respect to total households. Debhata is an upazilla in satkhira district with an area of 176.33 sq. km. It is bounded by satkhira sadar upazilla on the north,kaliganj upazilla on the south, kaliganj, assasuni and sastkhira sadar upazillas on the east, west Bengal of India on the west. Debhata consist of one mouza. It has an area of 3.48 sq. km. the town has a population of 2492. Male are 51.52%, female are 48.48%. literacy rate among the town people is 51.75%.main occupations are agriculture 26.58%,pisciculture 6.2%,agricultural labor 29.84%,service 4.28%, fishing 1.48% and others 6.79%.total cultivable land 20611 hectors, fallow land 2848 hectors. Average annual rainfall in Debhata is approximately 149.75 mm. The average temperature in Debhata is approximately ranging from 36.1 to 10.6 throughout the year. Morrelganj is an upazilla of bagerhat district in the division of Khulna. It consists of 9 wards and 12 mahallas. The area of the town is 15.36 sq. km. the town has a population of 22136. Male are 51.76%, female are 48.24%.the average literacy is49.5% & of which male literacy is 53.9% and female literacy is 40%.The main occupations are agriculture 35.49%, fishing 3.65%, agricultural laborer 20.73%, transport 1.4%.service 5.6%, others 14.43%.the cultivable land 31778.22 hectors. Average annual rainfall in Morrelganj is approximately 168.31 mm and the average temperatures in Morrelganj is approximately ranging from 35.8 to 12.2 throughout the year. Koira is an upazilla of Khulna district in the division Khulna. Koira has 7 wards, 72 mahallas and 131 villages. Total area is 1775.41 sq. km. males constitute 49.68% of population, and females are 50.32%. Koira has an average literacy rate of 32.4%.Average annual rainfall in Koira is approximately 155.56 mm and the average temperature in Koira is approximately ranging from 36.6 to 11.0 throughout the year. Labonchora is a mouza of Rupsha upazilla of the district Khulna. Rupsa has 5 Unions/Wards, 64 Mahallas, and 75 villages. Males constitute are 51.98% of the population, and females 48.02%. Labonchora has an average literacy rate of 40.4%.

2.4 Sample Size of Survey

Equation of Sample size formula for anthropometry in cluster design is

$$n^2 = [t^2 \times \frac{p \times q}{d^2}] \times DEFF \quad (7)$$

Where, n = sample size, t is linked to 95% confidence interval for cluster sampling (1.96), p is expected prevalence (fraction of 1), q is 1- p (expected non-prevalence),d is relative desired precision, DEFF = Design Effect. Based on a Sampling Methods and Sample Size Calculation for the SMART Methodology (June 2012) at the 95% confidence interval, ±10% precision, 50% prevalence, 1 and a design effect of 1.5 to account for cluster sampling. 150 households in each district were surveyed.

2.5 Household Survey Data

After completed questionnaire survey in four villages, household survey data was calculated in tabular form which is given below:

Table 11: Livelihood Vulnerability Index (LVI) sub-component values and minimum and maximum sub-component values for Debhata, Morrelganj, Koira and Labonchora, Bangladesh.

Major Component	**Sub-component**	**Units**	**Debhata**	**Morrelgang**	**Koyra**	**Labon chora**	**Max value**	**Min value**
Socio-demographic profile	Dependency ratio	Ratio	0.59	0.48	0.47	0.76	1.5	0
Major Component	Sub-component	Units	Debhata	Morrelgang	Koyra	Labon chora	Max value	Min value
	Percent of female-headed households	Percent	47	25	27	4	100	0
	percent of literacy	Percent	51.75	49.5	32.4	40.12	100	0
	Percent of households where head of household has not attended school	Percent	64	25	39	26	100	0
	Percent of households with orphans	Percent	3	0	0	0	100	0
Livelihood strategies	Percent of households with family member working in a community	Percent	66	43	58	65	100	0
	Percent of households dependent solely on agriculture as a source of income	Percent	36	33	13	1	100	0
	Average agricultural livelihood	1/#	0.51	0.68	0.58	0.49	1	0.1

	diversification index	livelihoods						
Social network	Average receive: give ratio	Ratio	2.20	1.67	1.33	1.5	8	0
	Average borrow: lend money ratio	Ratio	1.16	1.26	1.25	1.31	2	0.5
	Percent of households that have not gone to their local government for assistance in the past 12 months	Percent	63	73	65	53	100	0
Health	Average time to health facility	Minutes	22.39	19.99	41.63	15.83	60	1
	Percent of households with family member with chronic illness	Percent	50	65	63	37	100	0
	Percent of households where a family member had to miss work or school in the last 2 weeks due to illness	Percent	29	20	29	21	100	0
	Average malaria exposure*prevention index	Months*Bed net Indicator	4.88	5.72	3.09	3.69	8	0
Food	Percent of households dependent solely on family farm for food	Percent	21	6	8	0	100	0
	Average number of months households struggle to find food	Months	2.16	1.9	3.73	2.09	12	0
	Average crop diversity index	1/# crops	0.70	0.71	0.42	0.25	1	0.1
	Percent of households that do not save crops	Percent	66	78	73	38	100	0
	Percent of households that do not save seeds	Percent	78	75	92	53	100	0
	Percent of households reporting water conflicts	Percent	13	18	31	0	100	0
	Percent of households that utilize a natural water source	Percent	5	88	24	0	100	0
Major Component	**Sub-component**	**Units**	**Debhata**	**Morrelgang**	**Koyra**	**Labon chora**	**Max value**	**Min value**
Food	Average time to water source	Minutes	27.85	17.87	23.3	3.74	60	1
	Percent of households that do not have a consistent water supply	Percent	2	2	5	0	100	0
	Inverse of the average number of liters of water stored per household	1/Liters	0.024	0.016	0.032	0.04	1	0.005
Natural disasters and climate variability	Average number of flood, drought, and cyclone events in the past 6 years	Count	3.0	4.0	5.0	0	6	0
	Percent of households that did not receive a warning about the pending natural disasters	Percent	4	2	11	0	100	0
	Percent of households with an injury or death as a result of recent natural disasters	Percent	1	7	15	0	100	0
	Mean standard deviation of monthly average of average maximum daily temperature (years: 2001–2008)	Celsius	0.70	0.74	1.08	1.08	1.62	0.24
	Mean standard deviation of monthly average of average minimum daily temperature (years: 2001–2008)	Celsius	0.73	0.59	0.73	0.73	1.54	0.17
	Mean standard deviation of monthly average precipitation (years: 2001–2008)	Millimeters	70.97	81.64	85.87	85.87	253.38	5.04

3. Results

Based on survey data, results were prepared for major components of the LVI index such as Socio-demographic profile, Livelihood strategies, Social network, Health, Food, Water, Natural disaster and climate variability and the LVI-IPCC index to compare among four villages including the LVI sub-components value.

3.1 Survey Result on LVI Major Component

Table 3 presents the LVI sub-component values for each district as well as the minimum and maximum values for both combined. The major components and the composite LVI for each district are presented in Table 4. The dependency ratio index was higher for Labonchora (0.504) than Debhata (0.392), Morrelganj (0.320) and Koira (0.310). Overall however, Debhata (SDP 0.403) showed greater vulnerability on the Socio-Demographic Profile (SDP) index than Morrelganj (SDP 0.265), Koira (SDP 0.329) and Labonchora (SDP 0.241). Debhata respondents reported a higher proportion of female-headed households and a smaller proportion of household heads that attended school than koira, Morrelganj, and Labonchora respondents. The average reported age of Debhata female household heads was 24~56 years.

Debhata also showed greater vulnerability on the Livelihood Strategies component (0.491) than Morrelganj (0.469), Koira (0.414) and Labonchora (0.364). A higher percentage of Debhata households reported relying solely on agriculture for income (agriculture dependency index: Debhata (0.36) Morrelganj (0.33), Koira (0.13) and Labonchora (0.01).This is reflected in the livelihood diversification indices: the value of Debhata (0.452) is higher than Morrelganj (0.646), Koira (0.532) and Labonchora (0.432).

The Social Networks indicators were similar for the two districts. Over 63% of Debhata, 73% of Morrelganj, 65% of Koira and 99% of Labonchora households said that they had not approached their local government for assistance in the past month. Debhata households reported borrowing money more frequently and receiving more in-kind assistance from family and friends than koira, Morrelganj, and Labonchora households. 1.25 and 1.31 Labonchora; receive: give ratio: Debhata 2.20 Morrelganj 1.67, Koira 1.33 and Labonchora 1.5). Overall, Morrelganj households were more vulnerable than Debhata, Koira and Labonchora households on the Social Networks (SN) component of Debhata (SN 0.448) than Morrelganj (SN 0.481), Koira (SN 0.439) and Labonchora (SN 0.418).

Table 12: Indexed sub-components, major components, and overall LVI

Sub-component	Debhata	Morrelganj	Koira	Labonchora	Major component	Debhata	Morrelganj	Koira	Labonchora
Dependency ratio	0.392	0.320	0.310	0.504	Socio-demographic profile	0.403	0.265	0.329	0.241
Percent of female-headed house-holds	0.470	0.250	0.270	0.040					
percent of literacy	0.483	0.505	0.676	0.399					
Percent of house -holds where head of household has not attended school	0.640	0.250	0.390	0.260					
Percent of households with orphans	0.030	0.00	0.00	0.00					
Percent of households with family member working in a community	0.660	0.430	0.580	0.650	Livelihood strategies	0.491	0.469	0.414	0.364
Percent of households dependent solely on agriculture as a source of income	0.360	0.330	0.130	0.010					
Average agricultural livelihood diversification index	0.452	0.646	0.532	0.432					
Average receive: give ratio	0.275	0.208	0.167	0.188	Social networks	0.448	0.481	0.439	0.418
Average borrow: lend money ratio	0.440	0.503	0.502	0.537					
Percent of households that have not gone to their local Gov. for assistance in the past 12 months	0.630	0.730	0.650	0.530					
Average time to health facility	0.363	0.320	0.689	0.251	Health	0.440	0.471	0.499	0.418
Percent of households with family member with chronic illness	0.500	0.650	0.630	0.370					

Percent of households where a family member had to miss work or school in the last 2 weeks due to illness	0.290	0.200	0.290	0.210					
Average malaria exposure*prevention index	0.609	0.714	0.387	0.462					
Percent of households dependent solely on family farm for food	0.210	0.060	0.080	0.00	Food	0.499	0.485	0.479	0.250
Average number of months house -holds struggle to find food	0.180	0.154	0.311	0.174					
Average crop diversity index	0.664	0.682	0.352	0.167					
Percent of households that do not save crops	0.660	0.780	0.730	0.380					
Percent of households that do not save seeds	0.780	0.750	0.920	0.530					
Percent of households reporting water conflicts	0.130	0.180	0.310	0.00	Water	0.135	0.275	0.201	0.015
Percent of households that utilize a natural water source	0.050	0.880	0.240	0.00					
Average time to water source	0.455	0.286	0.378	0.046					
Percent of households that do not have a consistent water supply	0.020	0.020	0.050	0.00					
Inverse of the average number of liters of water stored per household	0.019	0.011	0.028	0.031					
Average number of flood, drought, and cyclone events in the past 6 years	0.498	0.665	0.832	0.00	Natural disasters and climate variability	0.260	0.289	0.406	0.224
Percent of households that did not receive a warning about the pending natural disasters	0.040	0.02	0.110	0.00					
Percent of households with an injury or death as a result of recent natural disasters	0.010	0.070	0.150	0.00					
Mean standard deviation of monthly average of average maximum daily temperature (years: 2001–2008)	0.335	0.361	0.610	0.610					
Mean standard deviation of monthly average of average minimum daily temperature (years: 2001–2008)	0.412	0.306	0.407	0.407					
Mean standard deviation of monthly average precipitation (years: 2001–2008)	0.265	0.309	0.325	0.325					
LVI						**0.365**	**0.374**	**0.388**	**0.242**

Chronic illness was reported by 50% of households in Debhata compared to 65% in Morrelganj, 63% in Koira and 65% in Labonchora. 29% of Debhata households said that a family member missed work due to illness in the past 2 weeks compared to 20% of Morrelganj households, 31% of Koira households and % of Labonchora households. Morrelganj households also reported being more vulnerable to malaria than Debhata, Koira and Labonchora households. When the sub-components were combined, the overall Health vulnerability (HV) score for Koira (HV 0.499) was higher than that for Morrelganj (HV 0.471), Debhata (HV 0.440) and Labonchora (HV 0.323).

Koira households reported struggling to find adequate food for their families 3.73 months per year on average compared to 2.16 months in Debhata, 1.9 months in Morrelganj and 2.09 months in Labonchora. Koira households reported growing 2.49 types of crops on average compared to 2.01 by Morrelganj households, 2.11 by Debhata households and 0.06 by Labonchora households. A higher proportion of Debhata than Morrelganj, Koira and Labonchora households reported storing crops and saving seeds (store crops index: Debhata 0.66, Morrelganj 0.78, Koira 0.73 and Labonchora 0.38; save seeds index: Debhata 0.78, Morrelganj 0.75, Koira 0.92 and Labonchora 0.53). The overall Food vulnerability (FV) scores for Koira (FV 0.479) was lower than that for Morrelganj (FV 0.485), Debhata (FV 0.496) and Labonchora (FV 0.414).

Labonchora also had a lower vulnerability score (0.015) for the Water component Debhata (0.135), Morrelganj (0.275) and Koira (0.201). In Morrelganj, 88% of households, 24% of households in Koira reported using a natural water source while more than 95% and 99% of households in Debhata and Labonchora reported getting water from a community pump. Similarly, Debhata households reported traveling 27.85 min on average to get water compared to 17.87 min in Morrelganj, 23.30 min in Koira and 3.74 min in Labonchora. 31% of Koira households reported hearing about conflicts over water in their communities compared to 13% of Debhata households, 18% of Morrelganj households and 0% of Labonchora households.

Both Debhata and Morrelganj districts had similar Natural Disaster vulnerability scores, based on the average reported number of flood, drought and cyclone events the past 6 years, the percent of households who said they received no warning, and the percent of households reporting a disaster-related injury or death. When climate variability was integrated into Natural Disaster index however, Koira households were more vulnerable (0.406) than Debhata (0.206) households, Morrelganj (0.289) households and Labonchora (0.224) households.

3.2 Composite LVI

Overall, Koira had a higher LVI than, Morrelganj, Debhata and Labonchora (0.388, 0.374, 0.365, 0.242 respectively), indicating relatively greater vulnerability to climate Change impacts. The results of the major component calculations are presented collectively in a spider diagram (Fig. 1). The scale of the diagram ranges from 0 (less vulnerable) at the center of the web, increasing to 0.5 (more vulnerable) at the outside edge in 0.1 unit increments. If we divided this scale into five sections named no effect, mild, moderate, severe, very severe. Then Debhata, Morrelganj and Koira lie between severe to very severe. Lobonchora is in moderate section. Fig. 3 shows that Koira is more vulnerable in terms of natural disaster and health profile, while Debhata is more vulnerable in terms of livelihood vulnerability and food profile, Morrelganj is more vulnerable in terms of water resources and social network profile.

3.3 LVI–IPCC

The LVI–IPCC analysis yielded similar results (LVI–IPCC: Debhata -0.06, Morrelganj -0.04, Koira 0.01 and Labonchora -0.02) If we consider a scale of 0 to 0.6 and divided it into low, medium, moderate and high contributing factor then Debhata, Morrelganj and koira is in moderate situation. However, accounting for the current health status as well as food and water security, Morrelganj may be more sensitive to climate change impacts than koira, Debhata and Labonchora (0.406, 0.385, 0.351 and 0.246 respectively). Based on demographics, livelihoods, and social networks, Debhata showed a higher adaptive capacity (0.437) than Morrelganj (0.378), Koira (0.356) and Labonchora (0.293). The overall LVI–IPCC scores indicate that koira (0.01) households may be more vulnerable than Debhata (-0.06) households, Morrelganj (-0.04) households and Labonchora (-0.02) households respectively.

4. discussions

4.1 Composite LVI Comparison

The major vulnerability components presented in Figure. 1 provide information on which household characteristics contribute most to climate change vulnerability in each district. Overall, Koira (0.388) had a higher LVI than Morrelganj, Debhata and Labonchora (0.374, 0.365, and 0.242, respectively). It indicates that Koira is more vulnerable to climate change. It is observed that the local governments of Morrelganj installed deep tube well in different place. These water management practices have likely decreased the vulnerability of the water sector in Morrelganj and are reflected in its low Water vulnerability score despite drought conditions. Similarly, although Koira households reported struggling to find food for almost 4 months longer per year than Debhata households, Morrelganj households and Labonchora households. . A smaller proportion of Debhata households reported engaging in seed storage and other food management practices. As a result, although Koira households did not report the same level of food insecurity as drought-stricken Debhata and Morrelganj households, the Debhata households had a higher vulnerability score. Morrelganj households also reported diversifying their income sources beyond farming by collecting natural resources to sell in the market and raising livestock such as cow, chickens. Despite these practices, Morrelganj was more vulnerable than Debhata,
Koira and Labonchora in terms of the Livelihood Strategies index. Although Morrelganj households reported a longer average time to health facilities and a higher prevalence of chronic illness, Koira households reported longer periods of malaria and lower rates of bed net ownership. We observed that a household that receives money or in-kind assistance often but offers little assistance to others is more insecure and vulnerable compared to those with excess money and time to help others. The finding that Lobonchora households had higher borrowed: lend and Debhata households had higher receive and give ratio may be related to the higher proportion of female-headed households in that district. Finally, although Koira households reported a higher absolute number of natural disasters over the past 8 years, the variability in the monthly average minimum and maximum daily distributed through local farming associations may help farmers time their plantings and prevent diversion of scarce water resources for irrigation.

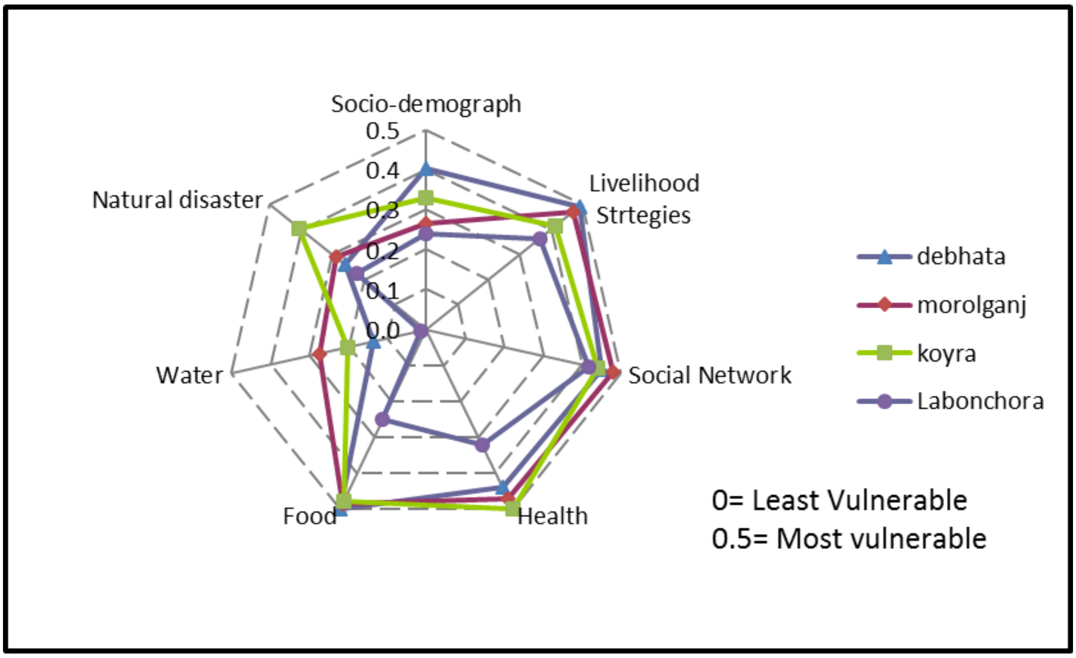

Figure 26: Vulnerability spider diagram of the major components of the Livelihood Vulnerability Index (LVI) for Debhata, Morrelganj, Koira and Labonchora

4.2 LVI-IPCC Comparison

Vulnerability triangle diagram of the contributing factors of the LVI–IPCC Index for Debhata, Morrelganj, Koira and Labonchora is shown in Figure 2. This plots the contributing factor values for exposure, adaptive capacity, and sensitivity of four villages. Despite the high estimated adaptive capacity of Debhata households resulting from demographic imbalance and adaptation practices such as livelihood diversification and food, natural disaster and water storage decreased Debhata's overall LVI–IPCC score. It is possible that these strategies will only be able to compensate for climate changes within a narrow band of possible climate variation. Although, Morrelganj, Koira and Labonchora households did not report similar adaptation strategies, they also did not report the same demographic pressures or low rates of school attendance prevalent in Debhata.

Figure 2 also shows that the overall adaptive capacity among four villages. Based on demographics, livelihoods and social networks parameters, Debhata showed a higher adaptive capacity than Morrelganj, Koira and Labonchora. Morrelganj is more vulnerable in terms of social networks parameters. The sensitivity among four villages. Accounting for the current health status as well as food and water security, Morrelganj is more sensitive to climate change impacts. On the other hand Labonchora is quite less sensitive to water security, food and health component.

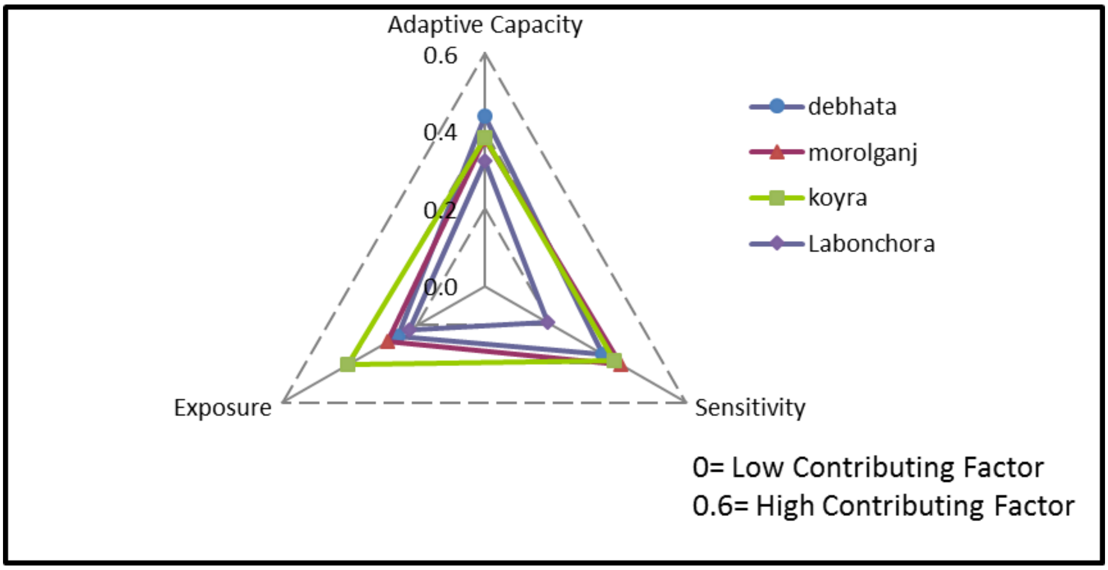

Figure 27 Vulnerability triangle diagrams of the contributing factors of the Livelihood Vulnerability Index-IPCC (LVI–IPCC) for Debhata, Morrelganj, Koira and Labonchora

5. conclusion

The Composite LVI and LVI–IPCC are two alternative methods for assessing relative vulnerability of communities to climate change impacts. Each approach provides a detailed depiction of factors driving household livelihood vulnerability in a particular region. Formulas for calculating the Composite LVI and LVI–IPCC were designed to be straightforward in order to reach a diverse set of users. Additional information can be gained when two or more study areas are compared using vulnerability spider and triangle diagrams. Limitations of our approach include the subjectivity involved in selecting sub-components and the directionality of the relationship between the sub-components and vulnerability, the masking of extreme values by utilizing means to calculate the indices, and possible selection bias due to empty households left out of the sample. Replication of this study in the same location over time might provide information about how the exposure, adaptive capacity, and sensitivity of districts change as adaptation practices are initiated. Future work might include refinement of the Social Networks sub-components in order to more accurately evaluate social bonds. Additionally, the Composite LVI approach could be tested at the community level in order to compare vulnerability among communities within a district. Overall, it is concluded that the Composite LVI can be a useful tool for development planners to evaluate livelihood vulnerability to climate change impacts in the communities in which they work and to develop programs to strengthen the most vulnerable sectors.

References

Boddiger, D. (2007). Boosting biofuel crops could threaten food security. Lancet , Vol. 370, pp. 923-924.

Chambers, R., Conway; G. (1992). Sustainable Rural Livelihoods: Practical Concepts for the 21st Century. Institute of Development Studies, UK.

CDP (2006). Situation analysis on Child rights violation. In Shrimp sector in the southwest coastal region of Bangladesh.

DHS (2006). Demographic Health Survey, Measure DHS: model questionnaire with commentary. Basic Documentation, Number 2.

Ebi, k., Kovats, R.S., Menne, B. (2006). An approach for assessing human health vulnerability and public health interventions to adapt to climate change. Environmental Health Perspectives. Vol. 114, pp. 1930-1934.

Government of Bangladesh (2008). Cyclone Sider in Bangladesh. Damage, loss and needs Assessment for Disaster recovery and reconstruction.

Hahn, M. B., A. M. Riederer, and S.O. Foster (2009). The Livelihood Vulnerability Index: A pragmatic approach to assessing risks from climate variability and change—A case studies in Mozambique. Global Environmental Change, Vo. 19(1), pp. 74-88.

Instituto Nacicnal de Estatistica Mocambique (National Institute of Statistics. Mozambique) (2007). Base de dados- Clima (Database-Climate). http://196.22.54.6/pxweb2007/Database/INE/01/13/13. (accessed on 23 December, 2007.)

IPCC (2007). Climate Change 2007: Impacts, Adaptation and Vulnerability. Contributing of Working Group II to the Fourth Assessment Report (Ch. 9). Cambridge University Press, Cambridge, UK.

Polsky, C., Neff, R., Yarnal, B. (2007). Building comparable global change vulnerability assessments: the vulnerability scoping diagram. Global Environmental Change, Vol. 17, pp. 472-485.

Sullivan, C. (2002). Calculating a water poverty index. World Development. Vol. 30, pp. 1195-1210.

UNDP (2007). Human development reports. http://hdr.undp.org/en (accessed on 25 December 2007).

UNEP (2004). Poverty-biodiversity mapping applications. IUCN World Conservation Congress, Bangkok, Thailand, 17-25 November 2004.

UNICEF (2008). MICS3 Manual. (Multiple Indicator Cluster Survey). United Nations General Assembly, 1997. S/19-2. Programme for the Further Implementation of Agenda 21. 19th Special Session, 23–27 June, New York.

USAID, 2007. Famine early warning systems network (FEWS-NET). http://www.fews.net (accessed on 24 December 2007).

WHO/Roll Back Malaria (2003). Economic impact of malaria: household survey. Williamsburg Emergency Management, 2004. Household natural hazards preparedness questionnaire. Peninsula Hazard Mitigation Planning Committee, Williamsburg, VA.

World Bank (1997). Survey in living conditions: Uttar Pradesh and Bihar. Household Questionnaire, December 1997- March 1998.

WFP (2007). Vulnerability analysis and mapping (VAM). http://www.wfp.org/operations/vam (accessed on 24 December 2007).

Appendix A

Calculating the health major component for the LVI for Koyra, Khulna

Sub-components for Health major component	Sub component values for Koyra	Max sub-component value for study population	Min sub-component value for study population	Index value for Koyra	Health major component values for Koyra
Average time to health facility (H_1)	41.63	60	1	0.689	0.499
Percent of households with family member with chronic illness (H_2)	63	100	0	0.630	
Percent of house -holds where a family member had to miss work or school in the last 2 weeks due to illness (H_3)	29	100	0	0.290	
Average Malaria Exposure*Prevention Index (H_4)	3.09	8	0	0.387	

Step 1: (repeat for all sub-component indicators):indexHealth_1Koyra = $\frac{41.63-1}{60-1}$ = 0.689

Step 2 :(repeat for all major components): $\text{Health}_{\text{Koyra}} = \frac{\sum_{i=1}^{n} index_{s_{d}i}}{n}$

$= \frac{H_{1\ Koyra}+H_{2\ Koyra}+H_{3\ Koyra}+H_{4\ Koyra}}{4} = \frac{0.689+0.630+0.290+0.387}{4} = 0.499$

Step 3: (repeat for all study areas): $\text{LVI}_{\text{Koyra}}$ =

$$\frac{(5)(0.259)+(3)(0.414)+(3)(0.439)+(4)(0.499)+(5)(0.479)+(5)(0.201)+(6)(0.406)}{5+3+4+3+5+5+6} = 0.377$$

Appendix B

Calculating LVI–IPCC for Koyra, Khulna

Contributing factors	Major components for Koyra	Major component values for Koyra	Number of sub-components Per major component	Contributing factor values	LVI–IPCC value for Koyra
Adaptive capacity	Socio-demographic profile	0.259	5	0.350	0.02
	Livelihood strategies	0.414	3		
	Social networks	0.439	3		
Sensitivity	Health	0.499	4	0.385	
	Food	0.479	5		
	Water	0.201	5		
Exposure	Natural disasters and climate variability	0.406	6	0.406	

Step 1 :(calculates indexed sub-component indicators and major components as shown in Appendix A, taking the inverse of the adaptive capacity sub-component indicators: Socio-demographic Profile, Livelihood Strategies, and Social Networks).

Step 2 :(repeat for all contributing factors: exposure, sensitivity, and adaptive capacity): Adaptive $\text{Capacity}_{\text{Koyra}}$

$= \frac{\sum_{i=1}^{n} W_{m_i} M_{id}}{\sum_{i=1}^{n} w_{M_i}} = \frac{(5)(0.259)+(3)(0.414)+(3)(0.439)}{5+3+3} = 0.350$

Step 3 :(repeat for all study areas): LVI $-\text{IPCC}_{\text{Koyra}} = (e_{Koyra} - a_{Koyra}) * s_{Koyra}$

= (0.406-0.350)*0.385=0.02

PREDICTING BANKLINE CHANGES IN THE KARNAFULI RIVER USING ARCGIS

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ABSTRACT
Using ArcGIS, river bank line and islands were digitized separately for each collected image from Landsat and the rate of resulting erosion and deposition were found as 334978.63 m^2/year 48230.39 m^2/year in 1989 to 2013. With due respect to different time span, the present resulting erosion rate is found as286748.24 m^2/year in 2013.Similarly, the area of inward islands decreased significantly during 1989 to 2002 and 2011 to 2013. Thus, this could be described as the river system has gained the sediment carrying capacity during study duration whereas a reverse situation observed during 2002 to 2011.This study also includes the bank line shifting near the river bank protection work at Sarafvata, in RanguniaUpazila. It has been observed that before the river bank protection work (i.e. construction period 2010 to 2012) the erosion rate was found as 3.43 m/year during2004 to 2012 whereas after river bank protection work (in 2013) the erosion rate was decreased to 2.73 m/year. With limited image analysis and field experience this is envisage that the transportation of sediment is the major contributing factor of bankline changes, however for concluding with a specific relationships among bankline change and riverbank protection works, continuous images are required.

Keyword:*Bankline changes; Riverbank protection; Images; ArcGIS; Karnafuli*

1. Introduction

Bangladesh has a large network of rivers, streams and canals with a total length of the river up to 50m width at least 24,000 km and an area of 4,600 km^2(Rahman et al., 2012).The flow processes continuously changes if the river is very dynamic. Active bank erosion and scouring is apprehended to change the scenario leading to loss of limited valuable land of Bangladesh. Due to geographical settings, Bangladesh use to suffer heavily floodplain flooding in spatial and temporal variation of monsoon rainfall (Rahman et al., 2012). Conventional river training have been practicing in Bangladesh from 1960s but the process is very expensive (Rahman et al., 2012).Details knowledge on river bank erosion and its protective measures are very important for sustainable river management. Changing pattern of the river bank largely depends on hydro morphological characteristics of the river (Rahman et al., 2012). River bank and bed scour associated with channel confluences and bends. Erosion plays an important role in the siltation process, and the water-holding capacity of rivers. Some researchers have reported that river bank erosion is taking place in about 94 out of 489 upazilas in Bangladesh (Rahman, 2010).

The process of erosion gradually destroys the shore lands / riverbanks, foreshore areas and successively the earthen embankments engulfing the plain agricultural lands, habitats and many important installations. In the affected reaches, the embankments are prone to disappear within a year or two due to erosion(Islam, 2011). Generally,new embankments are constructed along separate alignments, simultaneously adopting sufficient measures to check the bank erosion (Islam, 2011). After the river bank protection works the river morphology also changed besides the protection works(Rahman, 2010).In this study, morphology changes in Karnafuli River were assumed to carry outdue to riverbank protection works. The change of morphology before and after the river bank protection works was also expected to be continued.

2. STUDY AREA

Karnafuli River (Patenga to Kaptai) was taken as study area and then focused on Sarafvata in RanguniaUpazila (Fig. 1a and b).

The left bank of study area lies between latitude 22°18'51.75"N to 22°28'58.12"Nand longitude 91°49'18.66"E to 92°14'18.09"E and the right bank of study area lies between latitude 22°19'12.84"N to 22°28'54.98"N and longitude 91°49'0.45"E to 92°14'10.09". The length of Study Area (Karnafuli River) is about 85 km.

Figure 1a: Karnafuli River (Google Earth, 2013)

Sarafvata is one of the severe erosion prone deprived areas caused by river Karnafuli and Ichamati in which RanguniaUpazila is seriously affected. It has already devoured adjacent houses, shops, ponds, and valuable agricultural lands. The selected sites are highly erosion prone. Different types of river bank protection practices were evaluated in this site i.e.,Sarafvata in Rangunia Upazila, Chittagong.Geographically the riverbank protection work is situated within latitude 22°27'25.05"Nto 22°27'7.81"N and longitude 92° 2'14.77"E to 92° 2'44.31"E which indicated by point 1 and 2 in Fig 1bTotal distance of the river bank protection work is 950m.The left bank of study area lies between latitude 22°27'18.22"N to 22°26'45.09"N and longitude 92° 1'56.28"E to 92° 5'17.00"E which indicated by point 3 and 4 in Fig 1band the right bank of study area lies between latitude 22°27'24.41"N to 22°27'3.29"N and longitude 92° 1'51.32"E to 92° 5'10.91"E which indicated by point 5 and 6 in Fig 1bsituated at Rangunia upazila and village of Vumirkil show in (Fig1b). The length of this study area is about 6.10 km. From the total river courses only left bank considered for the entire study because major erosion had been occurred in the left bank and its adjacent area. This river bank protection work was started in 2010 and finished in 2012.The river bank protection work identified in (Fig 1b) by red color line.

Figure 1b: Sarafvata in Karnafuli River (Google Earth, 2013)

3. METHODOLOGY

3.1 Image Collection

A series of Landsat satellite images were downloaded for the Karnafuli River from www.glovis.usgs.gov such as image of 1989, 2002, 2011. Recent image for 2013 was collected from Google Earth (Table 1 and Fig 2).

Table 1: Details of data used in present study for Karnafuli River

Serial No	Types of data	Year	Source
1	Landsat MSS	1989	USGS
2	Landsat SLC	2002	USGS
3	Landsat TM	2011	USGS
4	Google Earth	2013	Google Earth

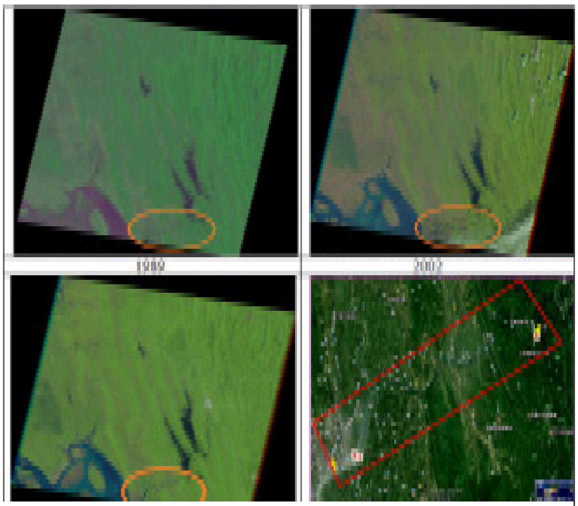


Figure 2: USGS, Google Earth satellite imagery of Karnafuli River

The relevent information on river bank line was extracted from the available Google Earth imagery in different time period, based on spatial overlays techniques. Newly developed historical imagery tool bar was used to collect satellite imagery in different time interval (year wise) by applying the sliding bar adjustment techniques. To maintain and fixed eye altitude of 5210.671 m and selecting whole study area, series of satellite imagery were acquired from Google Earth Pro of the year 2004, 2010, 2012, 2013.

3.2 Working Procedure of Software

In the present study, high resolution Google earth imagery and series of Landsat satellite imagery are considered for the delineation of the bankline of the study area. By using Google satellite imagery was used for this research to know the recent bank line shifting status compare to the previous USGS provided satellite imagery. Bank line was derived by demarcated and digitized accurately in between bank line and adjacent water features pixel, using certain precise zooming of pixel. Geo-referenced Landsat MSS, Landsat SLC Landsat TM were digitized first using ArcGIS. Image of Google Earth was digitized by drawing polygon in Google Earth where the bank line and water were adjacent .Then it was import in ArcGIS by using Xtools pro. Digitization was performed to collect shoreline or bank line layers from individual year basis. These images already have been georeferenced by geographic coordinate system. By using project tool these images was georeferenced by projected coordinate system and converted to shape file. To obtain actual result, these images (coordinate system) were modified by repair geometry. Then area of shape file was obtained. Island were derived by digitized the imagery where the water and island were adjacent in river. For calculating erosion and deposition, area of river with and without island in different years was calculated. To find out the erosion and deposition zone different shape file were overlaid. To show the bank line shifting, images are overlapped in one layer. By observing overlapped images, where erosion or deposition occurred was find out. To get the erosion and deposition area, at first one image was erased from another. After erasing, it divided (where two lines werecut by one anther) many small segments. To obtain each segment area, convert it multipart to single part. Then area of each small segment was obtained. These small segment areas indicate erosion and deposition.

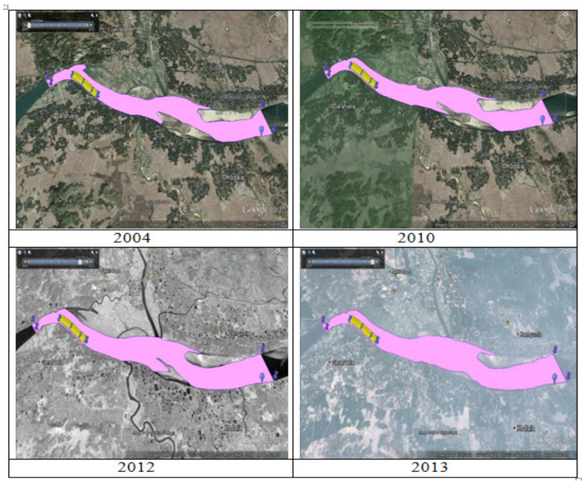


Figure 3: Google Earth satellite imagery of Karnafuli River focused Sarafvata

4. Findings

4.1 Combine Images of 1989, 2002, 2011 &2013

According to the erosion and deposition the study area (Karnafuli River) is divided into two zones. In Fig 4 red circle and green rectangle is indicating zone I and zone II respectively.

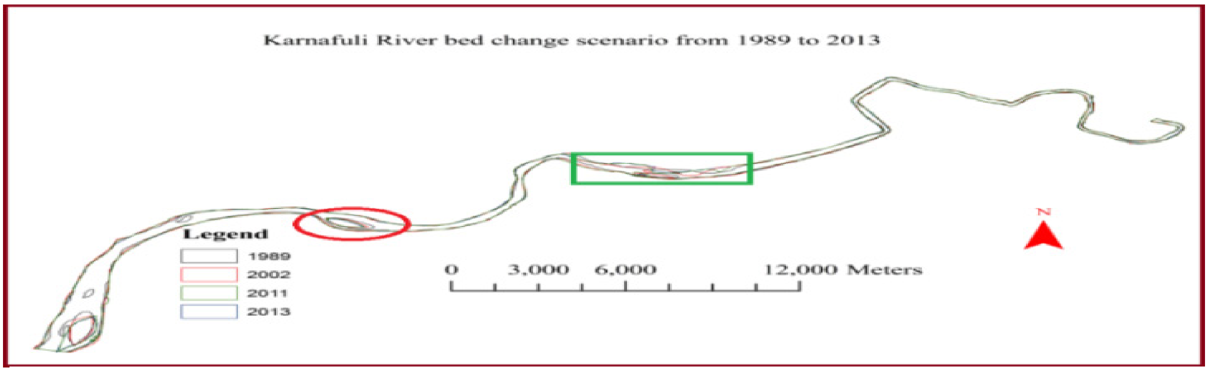
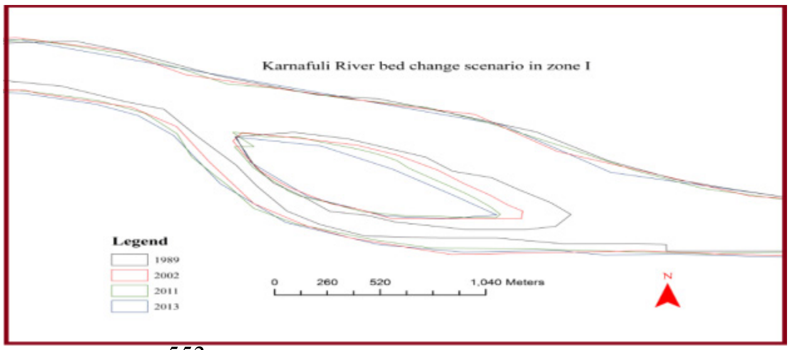


Figure 4: Karnafuli river bed change scenario from 1989 to 2013

Zone I: According to the calculated result and visual observation from the satellite imagery the zone I is high erosion zone. Fig 5 shows that high erosion occurs at left bank side at a considerable distance and low deposition occur at right bank side. It also shows the island has been eroded from year to year (Table 2).



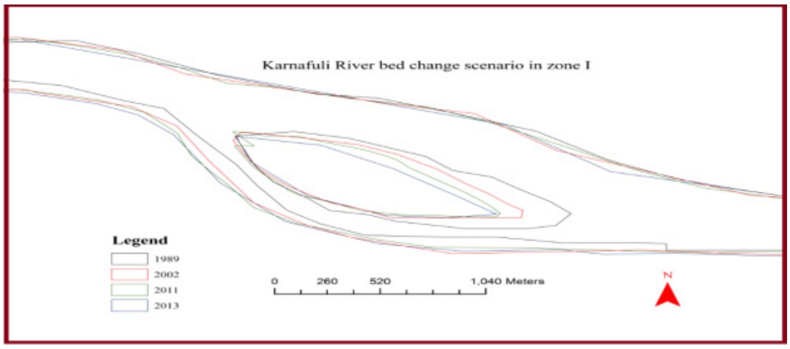


Figure 5: Karnafuli river bed change scenario in zone I

Zone II: Fig 6 shows that the left bank site at the starting of thezone has been eroded from year to year, but continuous deposition is occurred at right bank site from 1989 to 2013. At the middle portion of that zone,a severe deposition occurred at right bank site where the left bank has been deposited. The island is also eroded from year to year (Table 2). At the last portion of zone II the right bank site has been severe eroded and left bank site has been eroded slowly.

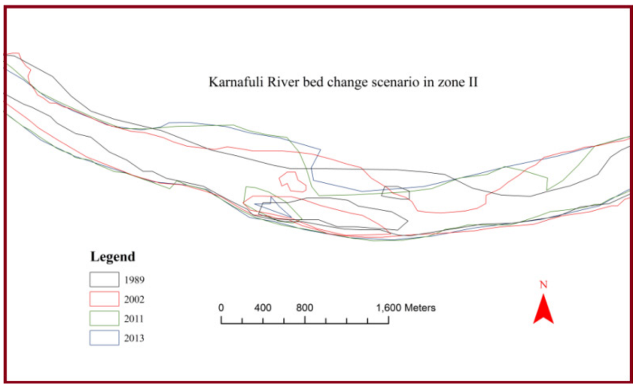


Figure 6: Karnafuli river bed change scenario in zone II

Table 2: Composite statistics of changing the area

	Island		River without Island		River with Island	
Year	Area (m^2)	Findings	Area (m^2)	Findings	Area (m^2)	Findings
1989-2002	822165.79	Erosion	2149440.61	Erosion	2971606.4	Erosion
2002– 2011	274963.64	Deposition	207340.27	Deposition	482303.9	Deposition
2011– 2013	292495.13	Erosion	47053.07	Erosion	245442.06	Erosion

For Sarafvata in Karnafuli River data of different superimposed images were considered for erosion and deposition area calculation. All the previous bank line shifting in different time periods were demarcated and calculated the erosion and deposition area. From the calculated results of different erosion and deposition, different types of erosion and deposition zone were identified and finally presented in the map to know the exact scenarios of the study area. Analyzing the tabular and graphical representations of the obtained values, erosion and deposition zone for the study area can be noted and marked with different color. From the visual interpretation with, it was clearly observed that the study area facing erosion and deposition in different spot. The erosion and deposition rate varied in different parts of the study area. The width of the channel was also varied from zone to lower zone. From the analysis and visual observation, category wise zone area identified on the basis of four period data. For showing the morphological change before and after river bank protection work, images were divided into two zones. Zone 1 shows the river bank protection site and zone 2 shows the nearest site of river bank protection work.

4.2 Before Protection Work

Comparison between 2004 &2010

Zone I: According to the calculated result and visual observation from the satellite imagery the zone 1 shows that low erosion occur at left bank side at a considerable distance and high deposition low erosion zone which found in right bank side. For this reason bank protection work was started at left bank in 2010(Fig 7).

Zone II: The zone 2 identified category shows that high erosion occurs at left bank side in a small length and low deposition zone which found in right bank side. But sum of erosion is higher than sum of deposition (Fig 7).

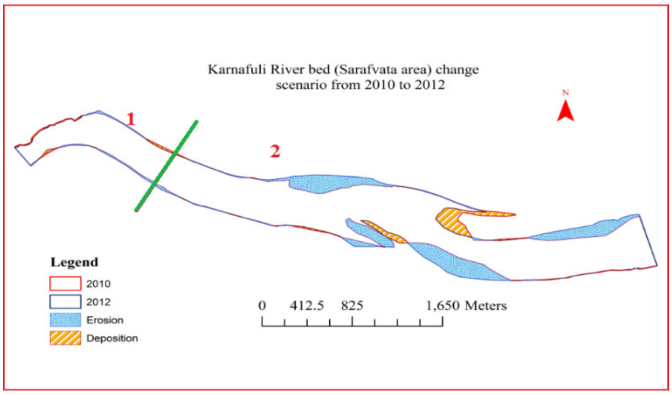


Figure 7: Erosion & Deposition of comparative year 2004 & 2010

4.3 After Protection Work

Comparison between 2010 &2012

Zone I: According to calculate result and visual observation from the satellite imagery the zone 1 shows that erosion and deposition rate is almost equal (Fig 8). This is expressing a positive result of river bank protection work.

Zone II: The zone 2 is currently facing severe erosion at left and right bank side and left and right bank side has fallen in low deposition zone category (Fig 8). Here also sum of erosion is higher than sum of deposition.

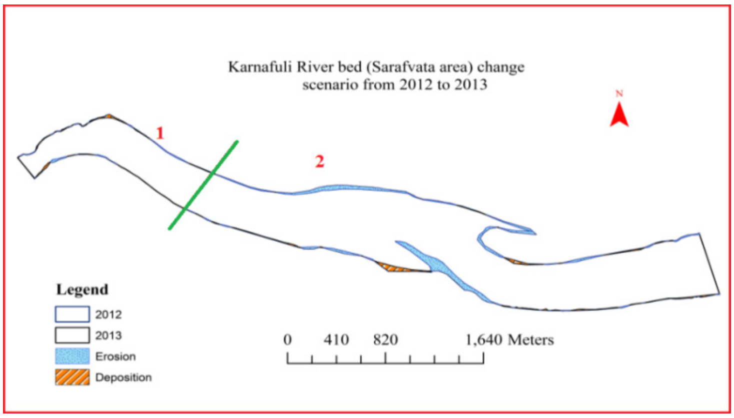


Figure 8: Erosion & Deposition of comparative year 2010&2012

Comparison between 2012 &2013

Zone I: According to calculated result and visual observation from the satellite imagery the zone 1 shows that erosion and deposition rate is almost equal. This is similar outcome as found in Fig 8, also a positive result of river bank protection work (Fig 9).

Zone II: The zone 2 identified category shows that lower deposition occur at left and right bank side and higher erosion zone which found in left and right bank side(Fig 9). Here also sum of erosion is higher than sum of deposition.

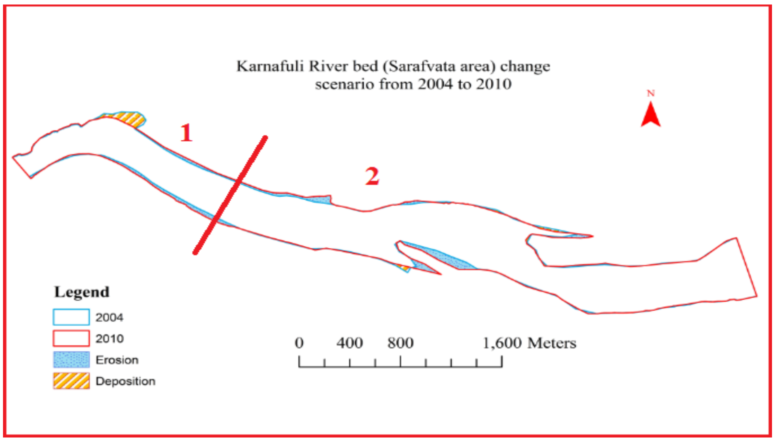


Figure 7: Erosion & Deposition of comparative year 2012&2013

5. Result

Analyzing the data it was found that the river area was increase from 25323573.68 m^2 to 27218620.96 m^2 in a span of 25 years from 1989 to 2013 (Table 3). The rate of resulting erosion was found as 334978.63 m^2/year during 1989 to 2013. On the other hand rate of the resulting deposition was 48230.39 m^2/year in 1989 to 2013 (Table 3). Thus, with due respect to different time span, the present resulting erosion rate is 286748.24 m^2/year in 2013 (Table 3). The area of inward Islands decreased significantly from 1989 to 2002 and 2011 to 2013 which means that the river system has gained the sediment carrying capacity over the years but from 2002 to 2011 it occurred inversely (Table 3).

Table 3: Composite statistics of Karnafuli River

Parameters	1989 (m^2)	2002 (m^2)	2011 (m^2)	2013 (m^2)
River area with Islands	25323573.68	27473014.29	27265674.03	27218620.96
River area without Islands	22607707.66	25579314.06	25097010.16	25342452.22
Area of Island	2715866.02	1893700.23	2168663.87	1876168.74
% River area increase/decreased (Base year 1989)	+8.4 %	+7.6 %	+7.4 %	

Most remarkable changes observed over the last 10 years period showing major left and right bank line shifting but very severe erosion observed during the 2004-2010 period(Fig 10). River is widening a lot in these zones due to developments of vast char(island) land near central part of the river which means huge volume of water hit in the left and right bank of Sarafvata. There after change detection was performed, demarcated and calculated the erosion and deposition of the study area from 2004-2013 and final layout were developed for visual interpretation and presentation which are showing in (Table 4) and (Fig 11).

Table 4: Sum of Erosion & Deposition of comparative year 2004-2013

Comparison between	Deposition (m^2)	Erosion (m^2)
2004&2010	29836.00	136918.40
2010&2012	110361.07	396861.70
2012&2013	32777.97	99221.19

Table 5: Digitized area of year 2004, 2010, 2012, 2013

Year	Area (m^2)
2004	2374558.3056
2010	2456819.6062
2012	2751545.7442
2013	2818071.9764

The changes show that erosion rate at Sarafvata was higher (396861.70 m^2) in the period of 2010 to 2012 than other two periods and the average rate of erosion in this period is 132287.23 m^2. Where as the period 2004-2010 and 2012-2013 shows the average erosion rate was19559.77 m^2/year and 49610.60 m^2/year respectively. The area of deposition was 110361.07 m^2 in 2010 to 2012 and but in the period of 2004-2010, it was about 29836.00 m^2 (Table 4).It has been observed that before the river bank protection work (i.e. considering construction period 2010 to 2012) the erosion rate was found as 3.43 m/year during2004 to 2012 whereas after river bank protection work in 2013 the erosion rate was decreased to 2.73 m/year. As a result, the severe erosion and deposition has been occurred in the Sarafvata area. But the rate of erosion is higher. The overall trend of the present left and right bank line was widening. For this reason the digitized satellite images gave the result of increasing area. Area of digitized images year 2004, 2010, 2012 &2013 are shown in Table 5

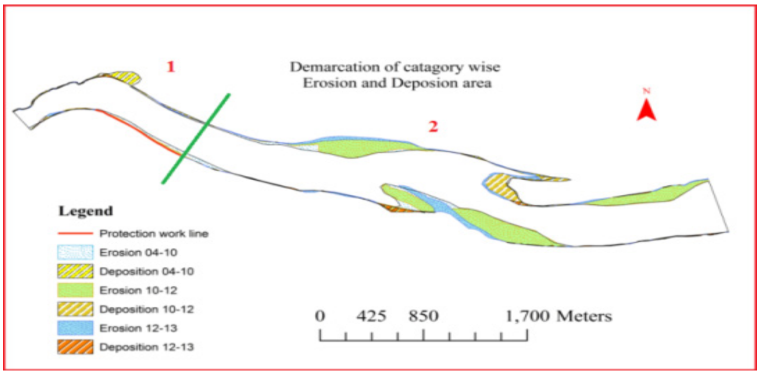


Figure 280: Erosion& Deposition of comparative year 2004-2013

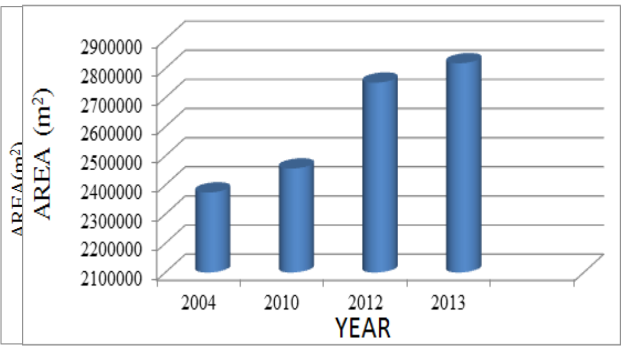


Figure 11: Erosion & deposition of comparative year of year 2004, 2010, 2012, 2013

6. Summaries

- From Sarafvata in Karnafuli River it had been seen that significant erosion was occurred from 2004 to 2012. But from Karnafuli River it had been seen that deposition was occurred from 2002 to 2011 that means river system lost sediment carrying capacity.

- Karnafuli River was eroded severely from 1989 to 2012 especially at left bank site in Sarafvata. So river bank protection works had been adopted in Sarafvata during 2010 to 2012.
- After the river bank protection the erosion rate is low compare to its previous erosion rate. But significant erosion is occurring near the river bank protection work both left bank and right bank. The erosion rate is higher than the rate of deposition. The bank line is moving in north direction in zone 1. But zone 2 shows north-eastward and south-eastward migration (Fig 13). When the erosion rate rises, deposition rate is also rises.
- River bank protection work in newly severe erosion zone area is needed to construct as early as possible. Latitude, longitude of starting and ending point and Length of protection work which need to construct is shown in table 6 and table 7. Figure 13satellite image shows the starting and ending.

Figure 13: Satellite view of highly eroded area

Table 6: Latitude & longitude of points shown in above figure

Point	Latitude	Longitude
1	22°27'14.84"N	92° 3'6.80"E
2	22°27'8.57"N	92° 4'10.31"E
3	22°27'1.58"N	92° 3'2.26"E
4	22°26'40.64"N	92° 4'44.05"E

Table 7: Protection work needed to construct

Bank site	Starting point	Ending point	Length of protection work (km)
Right	1	2	1.68
Left	3	4	3.06

References

Rahman, M.M, Arifur, R.M. and Munsur, R.M. (2012). "Effectiveness of River Training Structures in Bangladesh'', ICSE6 Paris - August 27-31.

Rahman, M .R.(2010). ''Impact of river erosion hazard in the Jamuna flood plain areas in Bangladesh", J. Sci. Foundation, Vol. 8(1&2), pp. 55-65, June-December 2010, ISSN 1728-7855.

Islam, M. N. (2011). "Embankment erosion control: Toward cheap and simple practical solutions for Bangladesh", Coastal Embankment Rehabilitation Project, South Khulshi, Chittagong, Bangladesh.

ASSESSMENT ON THE ADEQUACY OF NAVIGATION OF SOME INLAND WATERWAY ROUTES OF BANGLADESH

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ABSTRACT

Koratoa-Atrai-Gur-Gumani-Hurasagar River is situated in the Northwest region of Bangladesh. It is a tributary river of Ganges and Jumna River which faces problems of navigable depth, hinterland communication and low ability to hold water during the pre-monsoon and post-monsoon periods.The present study finds out the present depth and width of this river, checks the feasibility of navigation during the pre-monsoon and post-monsoon periods, determines the required channel depth and channel width for navigation and estimates the required dredging level at the selected stations.Morphological data, water level data and maps were collected from BWDB. For improving class III and IV,required channel depth is 3.62m for passing 1.52m draft vessel and required channel width is 33m for one-way traffic and 66m for two way traffic of 1.52m draft vessels. For improving class II categories, required channel depth is 4.23m andrequired channel width is 42m for one way traffic and 84m for two way traffic of 2.13m draft vessels. Maximum volume of dredging for developing class III and IV route is 198930 $m^3$and for developing Class II route its 210460 m^3.

Keywords: *Inland waterways, Navigation, River dredging*

2. INTRODUCTION

Bangladesh has around 800 rivers and since early times, inland water transport (IWT) had been the most important mode for internal trade in this country. The major rivers, together with their distributaries and feeder channels, provide an extensive natural highway for the movement of vessels ranging from large steamers to country boats of varied forms and sizes. Inland Water Transport (IWT) Systems have been used for centuries in countries including India, China, Egypt, the Netherlands, the United States, Germany, China, and Bangladesh. In the Netherlands, IWT handles 46% of the nation's inland freight, 32% in Bangladesh, 14% in the United States and 9% in China. Although water transport is slow, the physical environment of Bangladesh provides ample opportunities to utilize the numerous distributaries of the major rivers for the transport of bulky commodities.

The Master Plan of BIWTA recognized that the navigability of Bangladesh's waterways has deteriorated considerably over the last few decades. Sample studies of waterways in various regions of the country identified the following causes of deterioration: river siltation, reduction in trans-boundary flow, abstraction of surface- and ground-water, and reduction in tidal volume. Due to the above reasons, the rivers are gradually dying. As a result, it is hampering the smooth plying of watercrafts as well as irrigation facilities and fisheries production. Besides, it is creating obstruction to the drainage capacity of floodwater, which has adverse impact over the whole economy of the country.In this context, this research work focuses on finding out the adequacy of present depth and width of Koratoa-Atrai-Gur-Gumani-Hurasagar for navigation according to BIWTA. In this study, feasibility of navigation for this river during pre- and post- monsoon will be determined and also the required dredged volume will be estimated to attain adequate channel width and depth of selected river for navigation according to BIWTA.

3. STUDY AREA

Korotoa-Atrai-Gur-Gumani-Hurasagar River is located at the northwest part of Bangladesh. Satellite image of this river is shown in Figure 1.According to BIWTA Karatoa-Atrai-Gurnai-Gumani-Hurasagar River lays between class III and class IV.

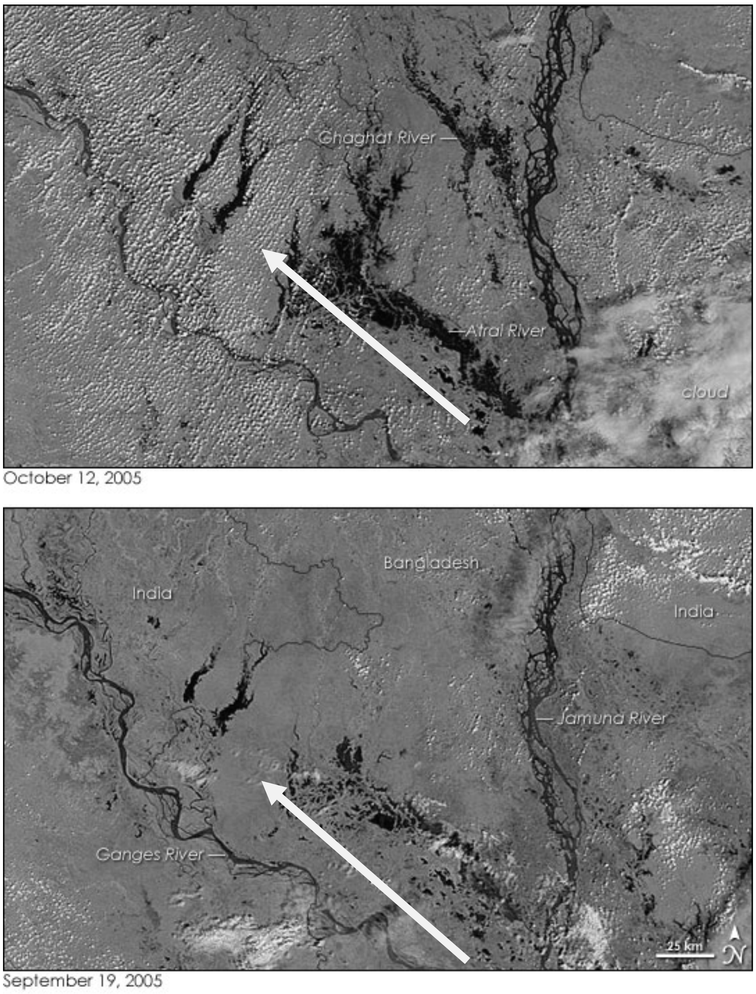

Figure 1: Korotoa-Atrai-Gur-Gumani-HurasagarRiver

4. METHODOLOGY

4.1 Data Collection

BWDB carried a hydrographic survey along the Korotoa-Atrai-Gur-Gumani-HurasagarChannel from Baghabari to Panchagarh during the period of 2004-2008(Table 1).These data include bank to bank width. These data have been collected from BWDB (2009) and reproduced.

Table 1: Description of Cross-Sectional Data.

River	Hydrological Region	Station ID	Location
Korotoa-Atrai-Gur-Gumani-Hurasagar	Northwest	RMBG1-12	Baghabari toChanchkair
Korotoa-Atrai-Gur-Gumani-Hurasagar	Northwest	RMKAG37-29	Chanchkair to Singra
Korotoa-Atrai-Gur-Gumani-Hurasagar	Northwest	RMKAG28-23	Singra to Atrai
Korotoa-Atrai-Gur-Gumani-Hurasagar	Northwest	RMBG22-17	Atrai to Mohadebpur
Korotoa-Atrai-Gur-Gumani-Hurasagar	Northwest	RMBG16-13	Mohadebpur toShamjhiaghat
Korotoa-Atrai-Gur-Gumani-Hurasagar	Northwest	RMKAG13-9	Shamjhiaghat to Khansama
Korotoa-Atrai-Gur-Gumani-Hurasagar	Northwest	RMKAG9-1	Khansamato Panchagarh

Water level data at stations from Baghabari station to Panchagar station for the year 2009 were collected from BWDB. Dimensions of different vessels available in Bangladesh were collected from BIWTA. Data on

Morphological network of Bangladesh and Hydrological station network of Bangladesh were taken from BWDB.

4.2 Data Analysis

First lowest water level and average water level have been observed from water level data of different station. Then month-wise comparison of lowest water level and average water level from January to December has been carried out. Finally, lowest low water level has been selected for every station.Second, water level in each cross section has been observed during dry monsoon. Then present width and depth of each cross section has been observed.Third, availability of each cross section has been checked for passing different vessels which is using in Bangladesh. Fourth, dredging requirement of each cross section has been estimated for 1.52m draft vessel (Class III and Class IV) and 2.13m draft vessel (Class II).

4.2.1 Month-wise Comparison of Lowest Water Level and Average Water Level

Figure 2 shows the monthly water level variation at Atrai station during the year 2009. Similar figures were prepared for other stations to compare monthly lowest water level and monthly average water level.

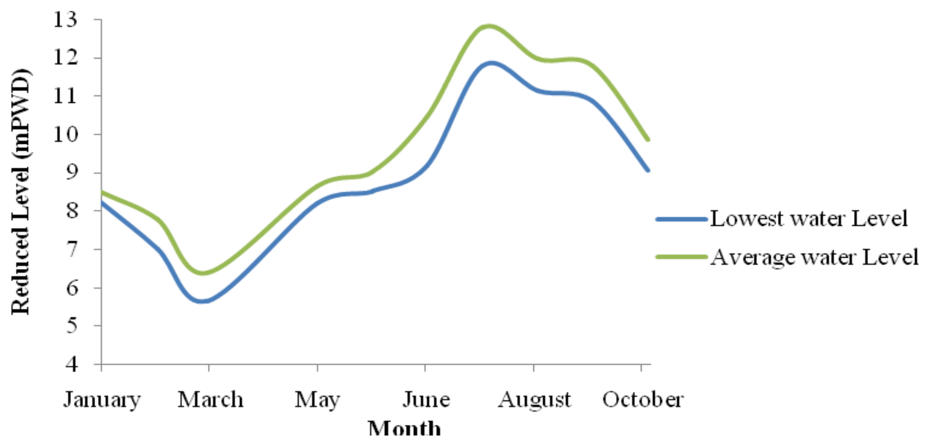

Figure 2: Monthly Water Level variation atAtrai station during the year 2009.

From the analysis it is found that, the lowest water level and the average water level at different stations differ slightly. During pre-monsoon and post-monsoon period, the value of lowest low water level and average low water level are almost same.During monsoon period, the value of lowest low water level and average low water level differ slightly. Table 2 shows the value of lowest low water and average low water at every station and also the difference between the two water levels.

Table 2: Lowest low water level and average low water level at selected stations.

Station Name	Station ID	Lowest Low Water Level (mPWD)	Average Low Water Level (mPWD)	Difference (m)
Bardeswari	SW139	82.78	82.87	0.09
Panchagarh	SW140	66.95	67.03	0.08
Khansama	SW142	41.23	41.34	0.11
Bhushirbandor	SW142.1	36.10	36.20	0.10
Shamjhiaghat	SW143	26.60	26.70	0.10
Mohadebpur	SW145	12.30	12.47	0.17
Atrai	SW147	5.65	6.39	0.74
Singra	SW147.5	6.76	7.14	0.38
Chanchkair	SW148	5.76	5.96	0.20
Astamanisha	SW149	2.38	2.53	0.15
GumaniRly.Bridge	SW149.1	3.32	3.34	0.02
Dohakoladanga	SW150	4.42	4.53	0.11
Baghabari	SW151	4.04	4.26	0.22

4.2.2 Observing Water Level at Cross-sections

Figure 3 shows the lowest low water level at a cross section during dry monsoon of 2009. Similar figures were prepared to observe the lowest low water level in each cross section during the dry monsoon in the year 2009.

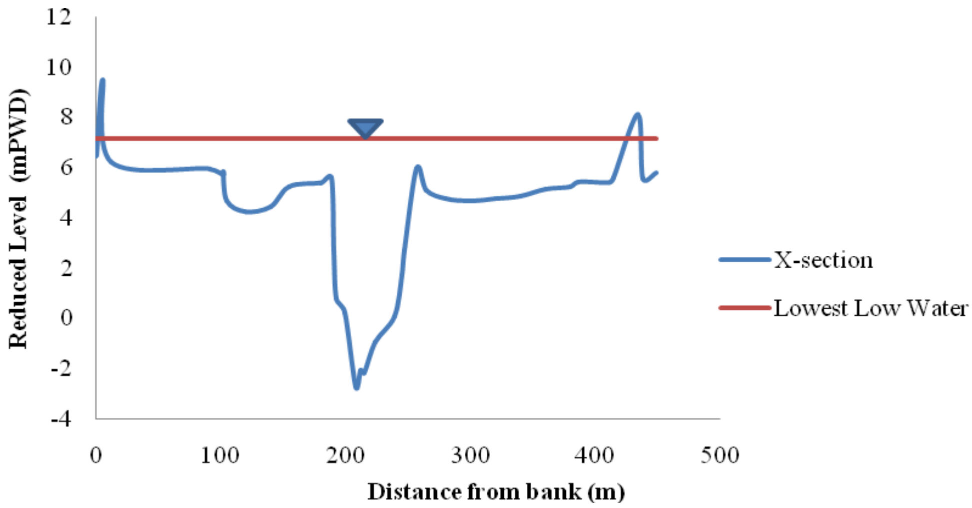

Figure 3: Lowest low water level at cross section RMKAG33 during the dry monsoon of 2009.

4.2.3 Determination of Required Drainage Level at Selected Stations

Figure 4 shows the required dredging level at station RMBG3. Similar figures were prepared to determine the required dredging level at all other selected cross sections during the dry monsoon in the year 2009.

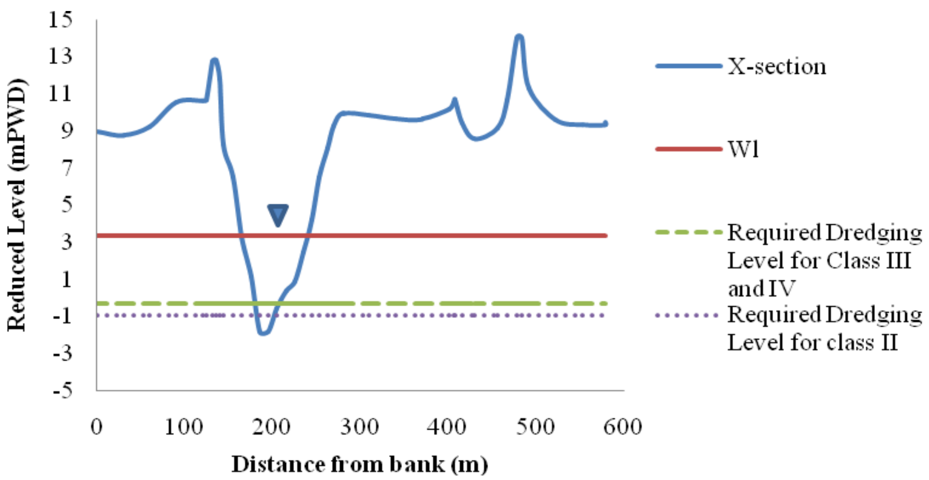

Figure 4: Required dredging level atstation RMBG3.

5. RESULTS AND DISCUSSION

5.1 Adequacy of Present Channel Depth and Width for Navigation

Koratoa-Atrai-Gur-Gumani-Hurasagar River has decayed during dry monsoon. The main cause of deterioration of this river is progressively increasing abstraction of both surface and ground waters in the dry season for irrigation purposes.

From the analysis it is found that, each station from RMBG1 to RMKAG37 has been deteriorated during pre-monsoon and post monsoon. Amount of water are inadequate for navigation. Depth of flow has been decreased. Width of each station has been minified. Many channels have been developed along cross section due to scarcity of water such as station RMBG1, RMKAG36 and RMKAG35. Station no RMKAG35 to RMKAG21 have much water than other station. Again station no. RMKAG9 has been deteriorated. Table 3 shows the present depth and width of the cross sections.

Table 3: Depth and width of cross-sections at different stations.

Station ID	Average depth of flow (m)	Top Width (m)	Comment
RMBG1	3.24	72	Not adequate for 1.52m and 2.13m draft vessel
	2.20	160	Not adequate for 1.52m and 2.13m draft vessel
RMBG2	2.55	96	Not adequate for 1.52m and 2.13m draft vessel
RMBG3	2.82	70	Not adequate for 1.52m and 2.13m draft vessel
RMBG4	1.13	60	Not adequate for 1.52m and 2.13m draft vessel
RMBG5	1.71	42	Not adequate for 1.52m and 2.13m draft vessel
RMBG6	1.97	63	Not adequate for 1.52m and 2.13m draft vessel
RMBG7	2.17	60	Not adequate for 1.52m and 2.13m draft vessel
RMBG8	1.60	42	Not adequate for 1.52m and 2.13m draft vessel
RMBG9	1.17	36	Not adequate for 1.52m and 2.13m draft vessel
RMBG10	1.02	35	Not adequate for 1.52m and 2.13m draft vessel
RMBG11	0.83	24	Not adequate for 1.52m and 2.13m draft vessel
RMBG12	0.82	15	Not adequate for 1.52m and 2.13m draft vessel
RMKAG37	1.26	63	Not adequate for 1.52m and 2.13m draft vessel

5.2 Feasibility of Navigation for the Selected River during Pre- and Post- Monsoon

According to BIWTA the river selected for this study lays between class III and IV. But presently, the depth and width of this river is regressively decreasing. As a result this river does not perform as class III and IV according to BIWTA during dry monsoon. That means 1.52m draft vessel cannot pass this river. Table 4 shows the feasibility of navigation at RMBG1 for different types of vessels.

Table 4: Feasibility of navigation at station RMBG1.

Name of vessels	Type of Vessels	Navigational Depth	Average depth of flow (m)		Required depth (m)		Comment
			Channe l 01	Channe l 02	Channe l 01	Channe l 02	
M V Parabot-9	Passenger	3.89	3.24	2.2	0.65	1.69	Need depth increase
M L Ocean Paradise	Passenger	4.24	3.24	2.2	1.00	2.04	Need depth increase
M V Kajol Dighi-3	Cargo	6.06	3.24	2.2	2.82	3.86	Need depth increase
M V Noiraj-5	Cargo	4.62	3.24	2.2	1.38	2.42	Need depth increase
O.T New Pacific	Oil tanker	4.29	3.24	2.2	1.05	2.09	Need depth increase
O.T Sarilia	Oil tanker	3.19	3.24	2.2	-0.05	0.99	Need depth increase
O.T Mubark-1	Oil tanker	3.17	3.24	2.2	-0.07	0.97	Need depth increase for channel 02 but channel 01 is suitable
M B Al Hamdulillah	Passenger	2.41	3.24	2.2	-0.83	0.21	Need depth increase for channel 02 but channel 01 is suitable
M B ForatNodi	Sand carrier	3.54	3.24	2.2	0.30	1.34	Need depth increase
M B Sakura Poribohon	Sand carrier	3.14	3.24	2.2	-0.10	0.94	Need depth increase for channel 02 but channel 01 is suitable
M B Dragger M S	Sand carrier Dragger	2.48	3.24	2.2	-0.76	0.28	Need depth increase for channel 02 but channel 01 is suitable

5.3 Required Dredged Volume to attain Adequate Channel Width and Depth for Navigation

Required depth and required width at the selected stations to support navigation for 1.52m and 2.13m draft vessels are shown in Table 5 and Table 6 respectively. The volume of dredged area is calculated for navigable both 1.52m draft vessel and 2.13m draft vessel. These are shown result in Table 7.

Table 5: Required depth and required width for 1.52m draft vessel.

Station ID	Allowable depth (m)	Allowable width (m)		Existing depth (m)	Existing width (m)	Required depth (m)	Required Width (m)	
		One lane	Two lane				One lane	Two lane
RMBG1	3.62	33	66	3.24	72	0.38	-39	-6
	3.62	33	66	2.20	160	1.42	-127	-94
RMBG2	3.62	33	66	2.55	96	1.07	-63	-30
RMBG3	3.62	33	66	2.82	70	0.80	-37	-4
RMBG4	3.62	33	66	1.13	60	2.49	-27	6
RMBG5	3.62	33	66	1.71	42	1.91	-9	24
RMBG6	3.62	33	66	1.97	63	1.65	-30	3
RMBG7	3.62	33	66	2.17	60	1.45	-27	6
RMBG8	3.62	33	66	1.60	42	2.02	-9	24
RMBG9	3.62	33	66	1.17	36	2.45	-3	30
RMBG10	3.62	33	66	1.02	35	2.60	-2	31
RMBG11	3.62	33	66	0.83	24	2.79	9	42
RMBG12	3.62	33	66	0.82	15	2.80	18	51

Table 6: Required depth and required width for 2.13m draft vessel.

Station ID	Allowable depth (m)	Allowable width (m)		Existing depth (m)	Existing width (m)	Required depth (m)	Required width (m)	
		One lane	Two lane				One lane	Two lane
RMBG1	4.23	42	84	3.24	72	0.99	-30	12
	4.23	42	84	2.20	160	2.03	-118	-76
RMBG2	4.23	42	84	2.55	96	1.68	-54	-12
RMBG3	4.23	42	84	2.82	70	1.41	-28	14
RMBG4	4.23	42	84	1.13	60	3.10	-18	24
RMBG5	4.23	42	84	1.71	42	2.52	0	42
RMBG6	4.23	42	84	1.97	63	2.26	-21	21
RMBG7	4.23	42	84	2.17	60	2.06	-18	24
RMBG8	4.23	42	84	1.60	42	2.63	0	42
RMBG9	4.23	42	84	1.17	36	3.06	6	48
RMBG10	4.23	42	84	1.02	35	3.21	7	49
RMBG11	4.23	42	84	0.83	24	3.40	18	60
RMBG12	4.23	42	84	0.82	15	3.41	27	69

Table 7: Required dredged olume of each station for Class III and Class IV, and Class II.

Station ID	Volume required (m^3)	
	Class III and Class IV	Class II
RMBG2	74230	159300
RMBG3	50510	53920
RMBG4	136360	144260
RMBG5	52810	116860
RMBG6	22030	24690
RMBG7	56520	63310
RMBG8	35400	38500
RMBG9	147090	157520
RMBG10	138380	147790
RMBG11	156680	165920
RMBG12	198930	210460
RMKAG37	28240	31320

A typical cross section after dredging is shown in Figure 5. Generally cargo vessel and oil tanker are unsuitable for passing this river due to its shallow depth. Mechanized country boat with 100-500 ton capacity can normally pass through this river. But there creates width scarcity. Also passenger vessels made of wood can be transferred by this river. Normally such vessels carry 15-25 people. Moreover sand carrier dredger with 0.70-0.92 m draft can pass through this river.

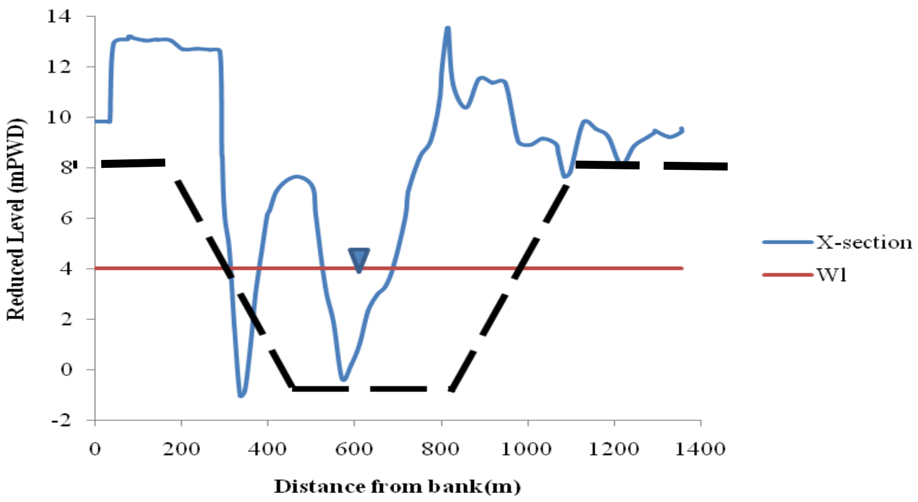

Figure 5: Typical cross-section after dredging at station RMBG1.

6. CONCLUSIONS

Following conclusions can be drawn from the foregoing analysis. From RMBG1 to RMKAG37 station of Karatoa-Atrai-Gur-Gumani-Hurasagar River, the depth varies between 0.83-3.24m. The width varies between 15-160m. The channel width at station RMBG1, RMBG2, RMBG3, RMBG4, RMBG5, RMBG6, RMBG7, RMBG8, RMBG9, RMBG10 and RMKAG37 are feasible for one-way traffic of 1.52m draft vessel (class III and IV). But the width at station no. RMBG11 and RMBG12 are not feasible for one way traffic of 1.52m draft vessel (class III and IV). The channel width at station RMBG1, RMBG2, RMBG3 are feasible for two way traffic of 1.52m draft vessel (Class III and IV). But the channel width atstation RMBG4, RMBG5, RMBG6, RMBG7, RMBG8, RMBG9, RMBG10, RMKAG37, RMBG11 and RMBG12 are not feasible for two way traffic of 1.52m draft vessel (Class III and IV). Depths of all the selected stations are not adequate for passing 1.52m draft vessel during dry monsoon. So, feasibility of passing 1.52m draft vessel (class III and IV) is not possible for above stations.

Required channel depth at all the selected stations is 3.62m for passing 1.52m draft vessel (class III and IV). Required channel width at all selected stations is 33m for one-way traffic and 66m for two way traffic of 1.52m draft vessels (class III and IV). For improving Class II categories, required channel depth at all the selected stations is 4.23m. Required channel width at all selected stationsis 42m for one way traffic and 84m for two way traffic of 2.13m draft vessels (class II). Maximum volume of dredging for developing class III and IV route is 198930 m^3. Maximum volume of dredging for developed Class II route is 210460 m^3. Based on the present study the following recommendations may be given for further study. A physical model can be performed to analyze river cross section and water level. Cost estimation of dredging volume can be conducted.Alternative way of developed navigable depth can be represented.

REFERENCES

Alam, A. (1976). Study of Navigability problems of selected River reaches and their solutions, B.Sc. Engineering Thesis, Department of Water Resources Engineering, Bangladesh University of Engineering and Technology.

BIWTA (1987). Master Plan, Chapter-5.6 Waterway Route Classification and Chapter-5.7 Navigational Standards.Bangladesh Inland Water Transport Authority.

BIWTA (1988).Master Plan, Draft Final Report, Volume 1, Part 1 & 2. Bangladesh Inland Water Transport Authority.

Bray, R.N., Bates, A.D. and Land, J.M. (1997). Dredging: A Handbook for Engineers, 2nd Edition, John Wiley & Sons Inc., New York.

Herbich, J.B. (2000). Handbook of Dredging Engineering, 2nd Edition, Macgraw Hill Publication.

Reza, A. and Rashid M. (2002). An overview of navigability of some selected rivers of Bangladesh, B.Sc. Engineering Thesis, Department of Water Resources Engineering, Bangladesh University of Engineering and Technology.

STUDY ON THE IMPLICATION OF THE RAIN WATER HARVESTING SYSTEM IN THE URBAN SLUMS & SCHOOLS

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ABSTRACT

Water and life represents the same value therefore inadequate access to safe water makes life worse. The condition is more than worse in developing country like Bangladesh where the demand of ground water is high. In Bangladesh access to safe water becomes a question in areas where physiographic, environment and other anthropogenic activities are complex. Dhaka is one of the most densely populated cities in the world. It is very difficult for a central water supply system to provide the required amount of water to all inhabitants of the city. The water level is being lower day by day. So an alternative system is needed to be introduced for meeting the local demand of water. As Bangladesh is a tropical country and receives heavy rainfall during the rainy season, rain water harvesting can be a solution to meet the public demand for water. Rain Water Harvesting (RWH) is an ancient practice of collecting rainwater and storing it for later use. It is less expensive, easy to construct, operate and maintain. The objective of this study is to give a brief idea about the post implementation situation of the rain water facility given in communities and schools. On the basis of primary data, the study was done and random sampling was followed in this survey. These data were collected from Bangladesh Adarsha Shikkha Niketan School Mirpur, Tilpa para Maleker Community, Hazi Ali Hossain School Mirpur, Meradiya Bhuiyapara Community, Mirpur Lalasarai community. It has been seen that regular operation & maintenance and the willingness of the users to use the rain water is the man driving force to keep operating the RWHS. Regular follow up can change the behavior of the target group. Mass campaign or raising awareness to change the behavior of the people in order to make rain water popular is requirement nowadays.

Keywords: *Central water supply system, Tropical country, Post implementation, Rain water harvesting system (RWHS), Target group*

1. INTRODUCTION

In most urban areas, population is increasing rapidly and one of the most urgent and significant challenges faced by decision-makers is to supply adequate water to meet societal needs and to ensure equity in access to water. With respect to the physical alternatives to fulfil sustainable management of freshwater, there are two solutions: finding alternate or additional water resources; or better utilizing the limited amount of water resources available in a more efficient way. To date, much attention has been given to the first option and only limited attention has been given to optimizing water management systems. Among the various alternative technologies to augment freshwater resources, rainwater harvesting and utilization is an environmentally sound solution, which can avoid many environmental problems often caused in conventional large-scale projects using centralized approaches.

Rainwater harvesting, in its broadest sense, is a technology used for collecting and storing rainwater for human use from rooftops, land surfaces or rock catchments using simple techniques such as jars and pots as well as engineered techniques. Rainwater harvesting has been practiced for more than 4,000 years, owing to the temporal and spatial variability of rainfall. It is an important water source in many areas with significant rainfall but lacking any kind of conventional, centralized supply system. It is also a good option in areas where good quality fresh surface water or groundwater is lacking. The application of appropriate rainwater harvesting technology is important for the utilization of rainwater as a water resource. The rainwater captured at source is considered as one of the purest water sources available. Rainwater quality always exceeds the surface water and comparable to that of ground water. The harvested rainwater does not come in contact with soil and rocks where it can dissolve salts and mineral which is harmful for portable and non-portable uses and at the same time not exposed to various pollutants that often transported with surface water into the river. The rainwater quality can be influenced by geographic location and economic activity in the area. The city dominated by heavy industry or localized industrial emissions may affect rainwater purity. Rainwater falling in rural and non-industrialized area can be superior to that in area dominated by heavy industrial and agricultural activities. It is essential that the rainwater harvesting system is planned; designed and constructed conforming to this guideline to ensure the discharged of the polluted first flush is taken care of. Rainwater is soft water compared to typical municipal tap water and not utilizing it as a supplementary water supply is a total waste of natural resources.

As Bangladesh has tropical monsoon with high rainfall and rain is the ultimate source of fresh water, so Rain Water Harvesting (RWH) system may function as a major alternative or supplementary source of water where the source is limited, inaccessible or unusual like iron and arsenic contamination of ground water or salinity problem in coastal areas of Bangladesh. Adaptation through rainwater collection may be particularly effective in tropical monsoon countries like Bangladesh, where the seasonal cycle in rainfall is large, (Pandey et. al., 2003). In some areas with high salinity problem, about 36 percent households have been found to practice rainwater harvesting in the rainy season for drinking purpose (Hussain, 2006).

Water Aid is an international non-government organization dedicated exclusively to the provision of safe domestic water, sanitation and hygiene education to the world's poorest people. Water Aid has been working in Bangladesh since 1986 to improve hygiene behaviour and access to water and sanitation services for poor communities giving emphasis on demonstration of innovative approaches, participatory methods, gender and vulnerable groups and sustainability. Water Aid is promoting sustainable, community managed safe water supply and sanitation facilities among the target population in rural areas and urban slums. It has many partner organizations. Those help them to establish and monitor several rainwater harvesting systems in different slums and schools. This study was mainly done by visiting two schools and three communities. The objective of this study is to find out the present condition of the existing RWHS established with the help of Water Aid Bangladesh for further improvement and to replicate more efficiently. Conducting a study on existing RWHS about its state of ability, usage, users' acceptance, operation & maintenance, management, monitoring/follow up are the specific objectives of the study.

2. RAINWATER HARVESTING SYSTEM (RWHS)

The origin of the term "water harvesting" is not known, but probably first used by the Geddes of the University of Sidney (Reddy, 2006). He defined water harvesting as "the collection and storage of any farm waters, either runoff or creek flow, for irrigation use." Several modifications of the definition have broadened the term to mean "the process of collecting natural precipitation from prepared watersheds for beneficial use." Different types of water harvesting system can be distinguished, including rooftop harvesting system, surface runoff harvesting and underground harvesting (IRC, 1992). Rooftop harvesting system is comprised of the rooftop as the catchment area, connected by gutters and pipes to the storage tank. The most suitable rooftop surfaces are corrugated iron sheets, although tiled; parachute cloth and asbestos sheet roofs can also be used. Surface harvesting systems catch rapid runoff from natural or man-made surfaces, then concentrate and store it strategic locations. Underground harvesting systems exploit water already infiltrated and concentrated through natural hydrological processes into the sand rivers that fill valleys in arid and semi-arid areas. In Bangladesh, different forms of water harvesting techniques are used in the hilly and flat areas. Hilly areas are located along the north-east borders of the country, which include the hills of Mymensingh, Sylhet, and Chittagong Hill Tracts (CHT). A typical Rainwater Harvesting System comprises of Roof catchment, Gutters, Downpipes, First Flush Pipe, Filter chamber, Storage tank. In a recent conducted study, many indigenous methods have been identified that are being used by tribal people of Bangladesh for water shed management including activities related to water and soil conservation, agro-forestry and religious rituals (Bose et. al., 1998). The rest of the country is largely a flood plain inhabited by people known as Bengali farmers have also been practicing various water harvesting methods for conserving water to meet household and agricultural demands.

3. METHODOLOGY

In this study, several steps are followed in methodology. The first step is the literature review about the RWHS and then sampling is done. The next step is the survey of the study area which consist Focus Group Discussion (FGD) and individual questionnaire. After that data compilation is done from the information find in the previous steps. The last step is analysis. The flow chart of the methodology is given in Figure 1.

3.1 Random sampling

Random sampling was followed in this survey. One of the best ways to achieve unbiased results in a study is through random sampling. Random sampling includes choosing subjects from a population through unpredictable means. In its simplest form, subjects all have an equal chance of being selected out of the population being researched. One of the biggest benefits of using random sampling in a survey is the fact that, since subjects are obviously randomized, it is the best way to ensure that results are unbiased. It is also much faster and often less expensive to use random sampling and as a result is a much more efficient way to obtain results. Additionally, random sampling consistently provides results that are valid, making it easy for researchers to draw conclusions about large populations. As with any survey, there is no way to guarantee that the results that come from a sample in a random survey are 100% accurate, although the results do tend to be more accurate than those obtained through other methods. The sample may not be representative of the larger population, which can incur a sampling error, but the chance of this occurring can be determined early in the survey by mathematical theories. Despite the problems associated with this method, it's important to remember that every survey comes with measures of uncertainty.

3.2 Survey The Study Area

After completing sampling randomly, the next step is to survey the study area. The study areas of the survey are Bangladesh Adarsha Shikkha Niketan School (Mirpur), Maleker Community (Tilpa para), Bhuiyapara Community (Meradiya), Hazi Ali Hossain School (Mirpur), Lalasarai community (Mirpur), Dhaka. To do that survey, two types of procedure are applied here, one is Focus Group Discussion (FGD) and another is individual questionnaire.

3.2.1 Focus Group Discussion (FGD)

A focus group discussion (FGD) is a good way to gather together people from similar backgrounds or experiences to discuss a specific topic of interest. The group of participants is guided by a moderator (or group facilitator) who introduces topics for discussion and helps the group to participate in a lively and natural discussion amongst them. The strength of FGD relies on allowing the participants to agree or disagree with each other so that it provides an insight into how a group thinks about an issue, about the range of opinion and ideas, and the inconsistencies and variation that exists in a particular community in terms of beliefs and their experiences and practices. FGDs can be used to explore the meanings of survey findings that cannot be explained statistically, the range of opinions/views on a topic of interest and to collect a wide variety of local terms. In bridging research and policy, FGD can be useful in providing an insight into different opinions among different parties involved in the change process, thus enabling the process to be managed more smoothly. It is also a good method to employ prior to designing questionnaires.

3.2.2 Individual Questionnaire

In the individual questionnaire step, people from schools and community were asked questions who are benefitted by the existing rainwater harvesting systems. They were asked about some basic information regarding the existing water supply system, prospects of RWHS, its advantage, disadvantage, improvement and many more.

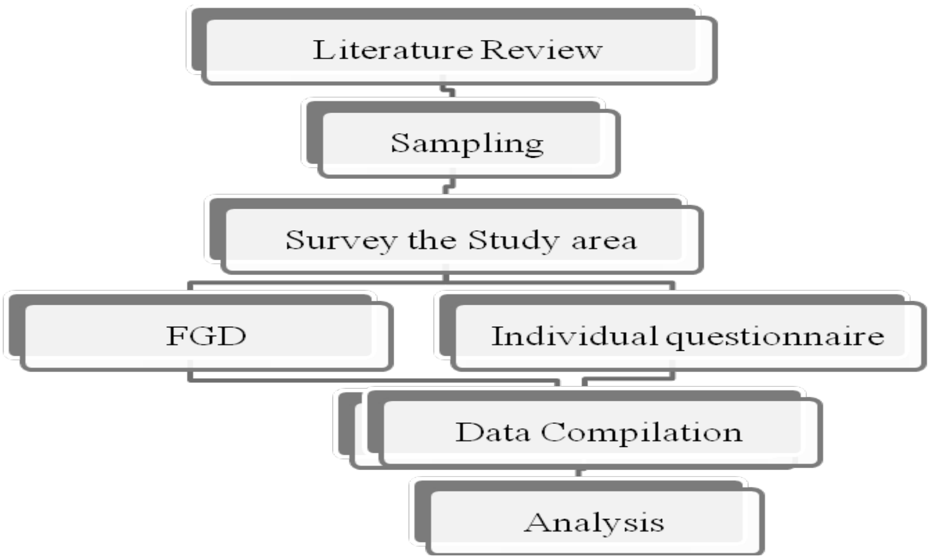

Figure 1: Flow Chart of Methodology

4. RESULT AND DISCUSSION

4.1 Result of the survey in Schools

After doing the individual questionnaire survey, it was found that their basic water requirement is 100 to 175 lpcd and they all use WASA water supply as their main water supply system. Some of them have regular water supply and some have intermittent water supply. People who have regular water supply facility consists 40% of the total people. Those who have intermittent supply of water, collect water once which is 40% or twice, which is 20% a day mainly at morning and evening. Water comes for 15 minutes to 1 hour per time. The percentage get from the survey about the water quality was 40% of them have odour free and clear water, 20% of them have odour and not clear water, 20% of them have odour free and not clear water & the rest 20% have odour and clear water (shown in figure 2). The purpose of their water use is domestic work like cleaning, cooking, washing, bathing etc.

According to this survey, the rate of people accepting rain water harvesting as another means of water supply is 80% which is highly satisfactory. The main cause of accepting this technology is the good quality of water. The main purpose of using rain water is mainly for drinking. From figure 3, it is clear that 20% people are not satisfied with the present rainwater harvesting condition because rainfall is not sufficient now. The main source of water supply in Dhaka city is Dhaka WASA. It is not possible for them to supply adequate and good quality water to all the inhabitants of this city. So it will be better if any new system can be found out to support our main water supply system. 60% people think that the caretaker does regular operation & maintenance (O & M), where the rest do not think similar. 80% people are interested to expand the present service condition (shown in figure 4) and 60% people (figure 5) agree to bear additional cost of O & M of RWHS. 20% people are not satisfied with the physical condition and cleanliness of the storage tank and the catchment.

The caretakers of the schools gave some information about the operation and maintenance of this system. They generally follow first flush regularly and wait minimum 15 minutes regarding first flush. In the rainy season, they harvest rain water minimum two times per day depending on the rainfall. They clean storage tank before every rainy season. They also repair catchments & gutters when required and clean them regularly. In this survey, mainly teachers, students and caretakers took part to find out the results. All the information from survey in schools is represented in table 1.

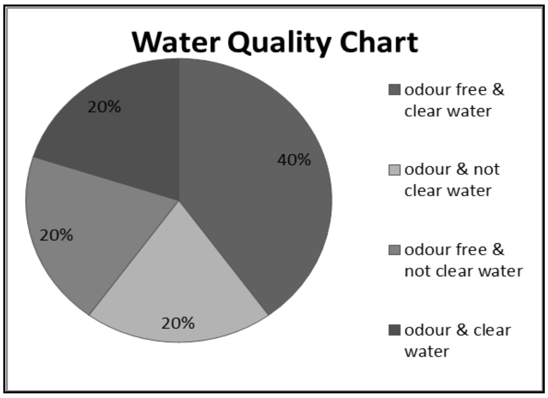

Figure 2: water quality chart of schools

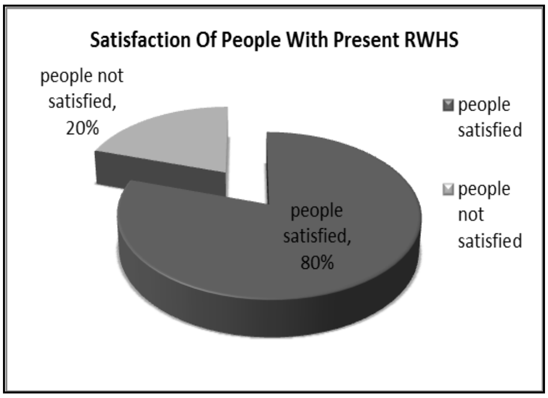

Figure 3: Satisfaction of people with present RWHS

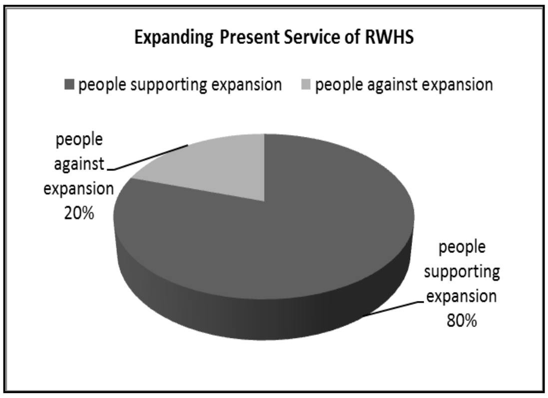

Figure 4: Expanding present service of RWHS

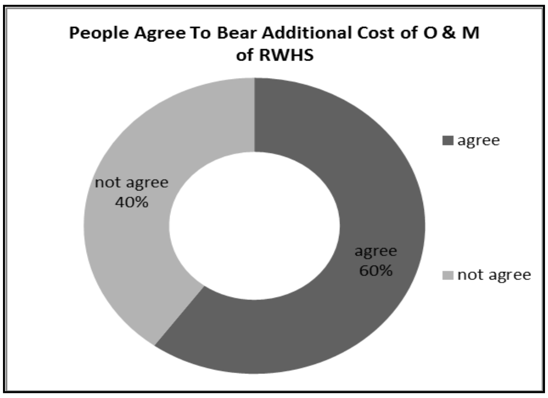

Figure 5: Agree to bear cost of O & M of RWHS

Table1: Information from survey in schools

Basic Information (Schools)						
Questions	Answer in Percentage		Answer in Percentage		Answer in Percentage	
1.Requirement of water (in LPCD)	100 liters	60%	175 liters	20%	30 liters	20%
2. Rate of collecting water	Once	40%	Twice	20%	Always available	40%
3. Quality of available water	Have odour but clear Odour free but not clear	20%	Have odour & not clear	20%	Odour free & clear	40%
Prospects						
Questions	Answer in Percentage				Answer in Percentage	
1.Rain Water Harvesting	Satisfactory		80%		Not Satisfactory	20%
2.Why rain water is satisfactory?	Good quality		80%		Others	20%
3.Possibility of harvesting rain water at night	Not possible		100%			
4.Regular O & M by Caretaker	Yes		60%		No	40%
5. Do you think present service condition could be expanded?	Yes		80%		No	20%
6. People agree to bear additional cost of O & M of RWHS	Agree		60%		Not agree	40%
Physical Status						
Questions	Answer in Percentage					
1.Condition of the catchment 2. Cleanliness of the catchment 3. Physical condition of the storage tank	satisfactory 80%				Not satisfactory 20%	
4. Does clean water come from the storage tank	Yes 100%					
Questions	Answers					
1.Follow first flush	Yes					
2. How long do you wait regarding first flush?	15 minutes					
3. Daily harvest during rainy season	2 times/ day					
4. Why rainwater is harvested usually at day time?	It is convenient					
5. Cleaning storage tank	Before rainy season					
6. repair catchment & gutters	When requires					
7. Repair storage tank	Never					
8. How often gutter and catchment is cleaned?	Regularly					

4.2 Result of the survey in Communities

Findings from the community survey are somehow different from the schools. Their requirement of water per capita is lower than the schools. It is mainly because of the type of water supply facility. There is no water supply from Dhaka WASA. So they use tube well as their main source of water. There basic water requirement is almost 60 to 90 lpcd. They collect water 2 to 3 times a day, usually at morning and afternoon. As they use tube well water so 100% of the water is clear and free from odour.

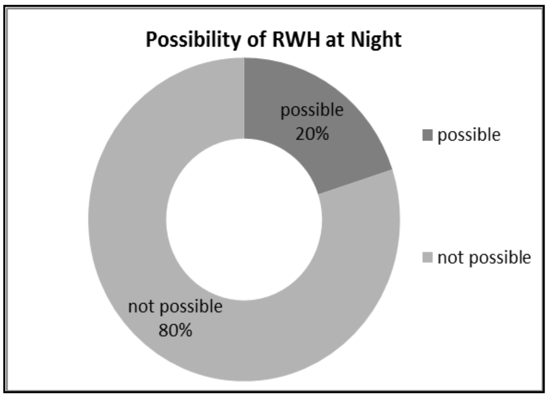

Figure 6: Possibility of RWH at night

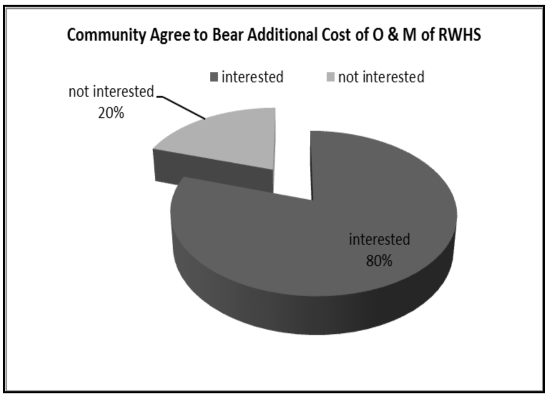

Figure 7: Agree to bear cost of O & M of RWHS

The use of rainwater is mainly for sanitation. So the storage tank is placed near the latrines so that they can use rain water for sanitary purpose. All people think that RWHS is satisfactory at their community because quality of water is satisfactory. 20% people think it is possible to harvest rainwater at night but a large portion actually 80% people do not find any feasibility to harvest rainwater at night (shown in figure 6). There is a difference between community people and people from schools. In school, rainwater is mainly harvested by caretaker but in communities all people participate in harvesting water though they have separate caretakers. They clean storage tank once in a week generally. From figure 7, it is clear that 80% people are interested to bear additional cost of O & M of RWHS. The generally use plastic tank as storage. They clean catchment, gutter regularly and repair them when needed. All the information from survey in communities is represented in table 2.

Table 2: Information from survey in communities

Basic Information (Communities)						
Questions	Answer in Percentage		Answer in Percentage		Answer in Percentage	
1.Requirement of water (in LPCD)	90Liters	80%	60 Liters	20%		
2.Rate of collecting water	Twice	40%	Thrice	20%	More than thrice	40%
3.Quality of available water	Odour free & clear	100%				

Prospects				
Questions	Answer in Percentage		Answer in Percentage	
1.Rain Water Harvesting	Satisfactory	100%		
2.Why rain water is satisfactory?	Good Quality	100%		
3.Possibility of harvesting rain water at night	Possible	20%	Not Possible	80%
4. Who clean the storage tank?	Everyone	80%	Caretaker	20%
5. When do they clean storage tank?	Once in a week	60%	Regularly	40%
6. Communities agree to bear additional cost of O & M of RWHS	Interested	80%	Not Interested	20%

Physical Status of RWHS				
Questions	Answer in Percentage		Answer in Percentage	
1.Cleanliness of the catchment	Clean	60%	Not Clean	40%
Questions	Answers			
1.Physical condition of the gutter	Not broken			
2. Type of storage tank	Plastic			
3. Physical condition of the storage tank	Satisfactory			
4. Does clean water come from the storage tank?	Yes			
5. Cleaning catchment	Regularly			
6. Cleaning gutter	Regularly			
7. Cleaning storage tank	Often			
8. Repair catchment	When requires			

5. RECOMMENDATIONS

To maintain RWHS properly operation & maintenance is must. Without O&M no RWHS can be successful. Based on the analysis and field experience following recommendations are given to run RWHS properly.

1) Take necessary action to raise awareness among people regarding uses of rain water
2) Revolving fund might be developed to continue the repair work
3) Willingness of user is an another drive to keep run the RWHS
4) First flush should be maintained properly
5) Gutter and catchment should be kept clean regularly and repair when needed
6) Regular O & M is essential

6. CONCLUSION

Rain is the best source of pure water without any treatment. More awareness should be made to make Rain Water Harvesting a success. Special monitoring is needed to operate this system successfully. If students, teachers, and community people become interested and Government and NGOs are willing to work on it, then it will become a great success at our country.

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REFERENCES

Bose, S.K., Gani, O., Hossain, A. T. M.E., Mridha, N.N. and Muhammad, T. (1998). “A Completion of Indigenous Technology Knowledge for Upland Watershed Management in Bangladesh”, Participatory Watershed Management Training in Asia (PWMTA) Program, GCP/RAS/161/NET-RAS/93/062, FAO (UN), UN Building, P.O.Box25, Kathmundu, Nepal.

Hussain, M. M. (2006). “Rainwater Harvesting Tanks for Supplementing Minor Irrigation Tanks During Drought”, National Seminar on Rainwater Harvesting And Water Management 11-12 Nov. 2006, Nagpur.

IRC (1992), “Water Harvesting: a guide for planners and project managers”, International Research Center, Technical Paper Series 30

Pandey, D.N., Gupta, A.K. and Anderson, D.M. (2003). “Rainwater harvesting as an adaption to climate change”, Current Science, Vol.85, No.1, 10 July 2003.

Reddy,Y.A. (2006). “Water Harvesting :Limitations in Implementation”, National Seminar on Rainwater Harvesting and Water Management, Nagpur, 11-12 Nov. 2006

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MORPHOLOGICAL CHANGE IN THE OPEN CHANNEL FLUME DUE TO THE EFFECT OF BAMBOO BANDALLING STRUCTURES

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ABSTRACT

River bank erosion is one of the key issues as a natural disaster in Bangladesh. Due to the river bank erosion, the river morphology is changed. In this paper, a laboratory study result is presented. In the laboratory open channel flume is used for the right bank erosion protection with the bamboo bandalling structures. A series of these structures are constructed on the right bank. It is seen that the laboratory open channel flume river bank is deposited behind the bamboo bandalling structures. Due to the effect of the bamboo bandalling structures, near bank velocity is reduced & so that the river bank is deposited with the water born sediment materials. River bank deposition indicates the erosion protection and at the same time the river morphology is changed.

Keywords: *River bank, Erosion, Protection, Laboratory; Flume; Morphology.*

1. INTRODUCTION

Experimental studies are effective ways of understanding of the physics of the flow in the channels. The controllable experimental parameters make laboratory measurements of significant meaning. The flume was 22m long, 2.2m wide and 1m deep. The bottom of the flume was filled with sand and a drainage pipe was provided at the bottom of the channel so that the channel made quasi-alluvial condition. The sketch of the experimental flume was depicted in Figure 1 as well as Figure 2. It is noted that previously a small scale flume study conducted by rahman, et al. (2004).

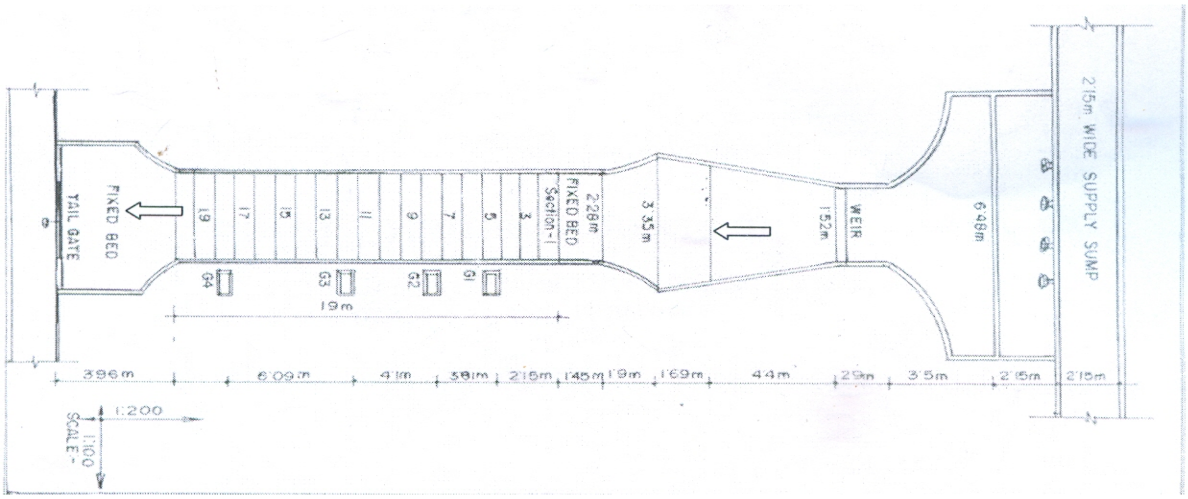

Figure 1: Layout plan of the experimental set-up

Figure 2: Experimental–setup with the full scale water supply system

2. METHODOLOGY

The flume was divided into a deep pool section, an upstream fixed bed section, a middle test section and a downstream fixed bed section. In the upstream fixed bed section, the bed is fixed with sand cement plastering. It is situated 1.05 m above the flume bed and extended 5m downstream from the upstream deep pool of water. The test reach was composed of a movable bed with relatively fine and uniform sands to a depth of 0.45m. The bed slope of the mobile section channel is 0.05m/19m. There was a downstream reach which is fixed sand cement plastering. It is situated 1.00m above the flume bed. The elevation difference between the upstream and downstream fixed bed is 0.05m. At the outflow point of the channel, a tailgate has been set to control the water level. The water passed over a sharp crested weir before it flowed to a small reservoir from which the pumped water come to the test flume. Near the sharp crested weir, the water supply system is connected to the still water

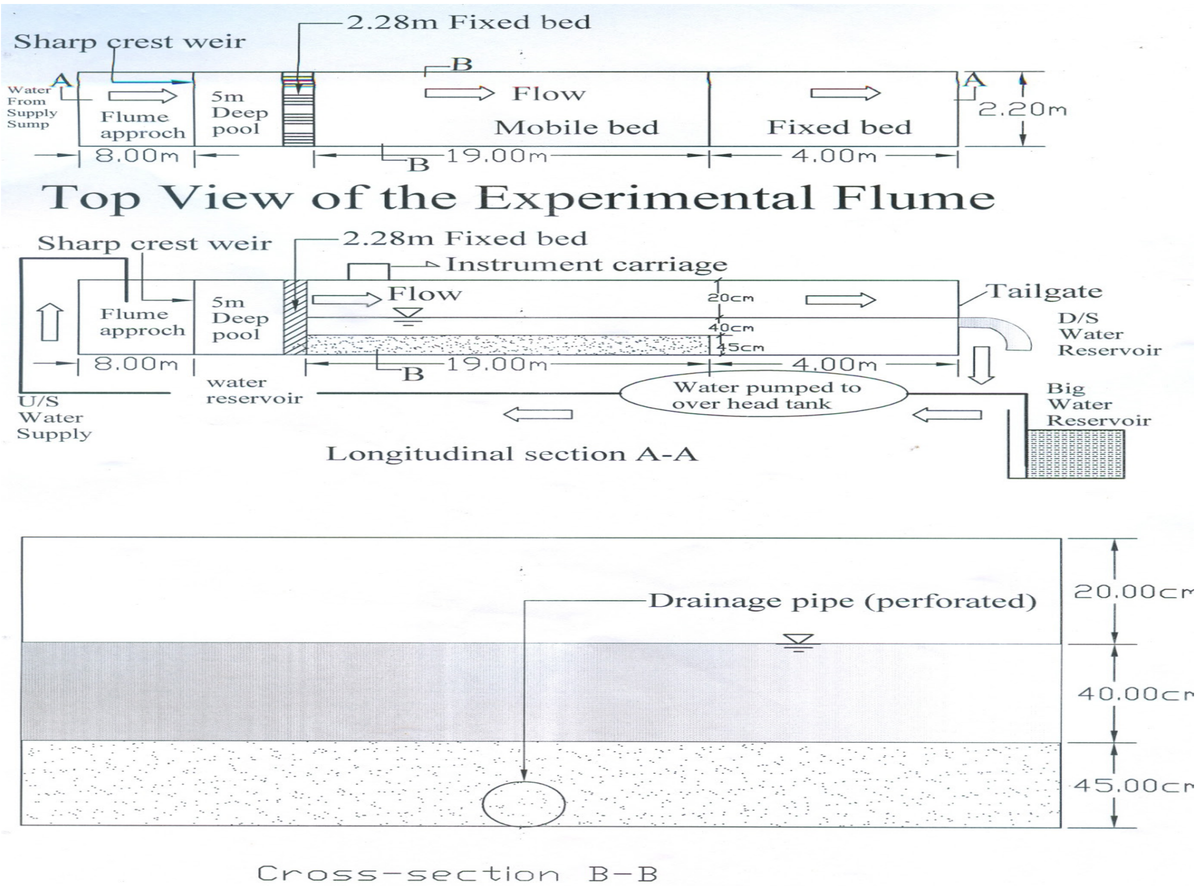

Figure 3: Different components of the experimental set-up

gauge and the discharge might be controlled manually. It is mentioned here that the flowing water from the test flume come to the downstream water reservoir from which water is pumped back to the laboratory water supply system as in Figure.3.

2.1 Fabrication of the bamboo bandalling structures

Along the right side of the test flume, bamboo bandalling structures have been equipped in the corresponding experimental run. The bamboo bandalling structures are made from the bamboo sticks and bamboo chatai. The dimensions of the bamboo bandalling structures are shown in Figure. 4.

Figure 4: Photographs of bamboo bandals ready for using in the test runs of flumes

When bandals placed in the laboratory flume, there was opening below the bamboo chatai by 50% of water depth. It is noted that the water depth 36 cm was fixed which measured in the upstream fixed bed and it is seen as in Fig. 5.

2.2 Sediment properties

The sediment used for the experiments had a median diameter of 190 micro meters which was based on the sieve analysis. The sieve analysis results were shown in Figure 6.

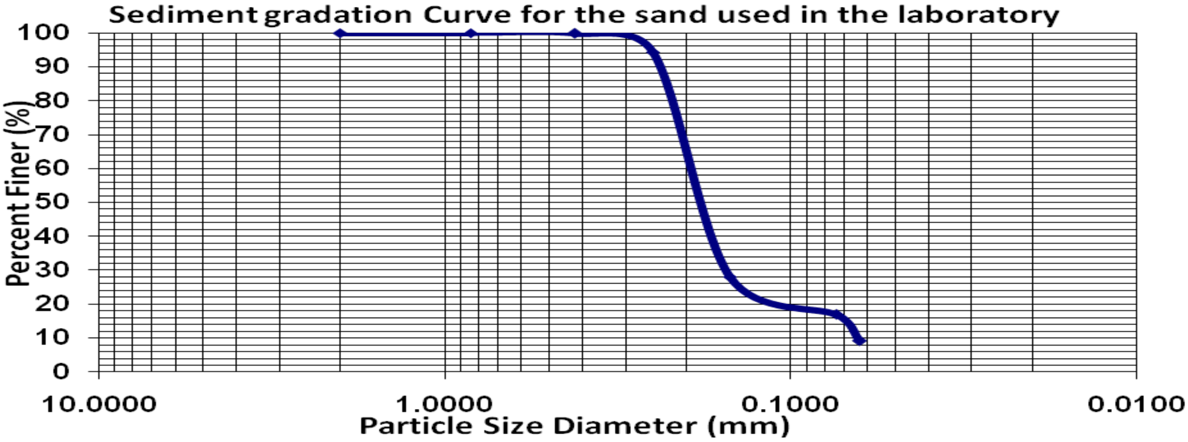

Figure 6: Grain size analysis of the sand used in the laboratory experiment

The angle of repose for the sand sediment was measured to 32° by taking the sample from the channel bed after experiments completed. The sand sediment used in the laboratory whose density was 2.65 g/cm^3, the critical shear velocity was calculated 1.13 cm/s where as the sand sediment falling velocity was estimated 3.14 cm/s from the laboratory experiment.

2.3 Data measurement and analysis

The 2D flow velocity and the bed deformation were measured at equilibrium state. The velocity components were collected with the electromagnetic velocity meter (model ACM250-A, Alec Electronics Co.,Ltd.) under dynamic flow conditions. The velocities were recorded by means of electromagnetic velocity meter.. The measurement device of the electromagnetic velocity meter was mounted on an instrument carriage that travelled on the rails over the channel. Point gauges were there which were utilized to measure the water level. The float tracking of the flow velocity on the free surface was visualized before stopping the pump by employing paper chips and floating ball as a float.

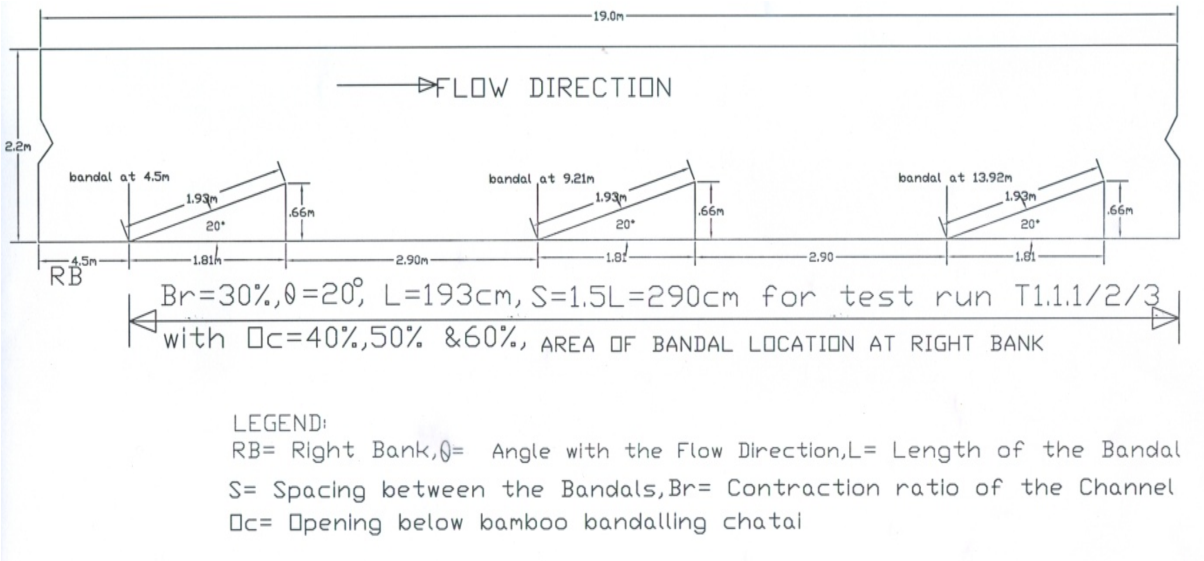

Figure 7: Experimental arrangement of the bamboo bandalling structures in the laboratory flume

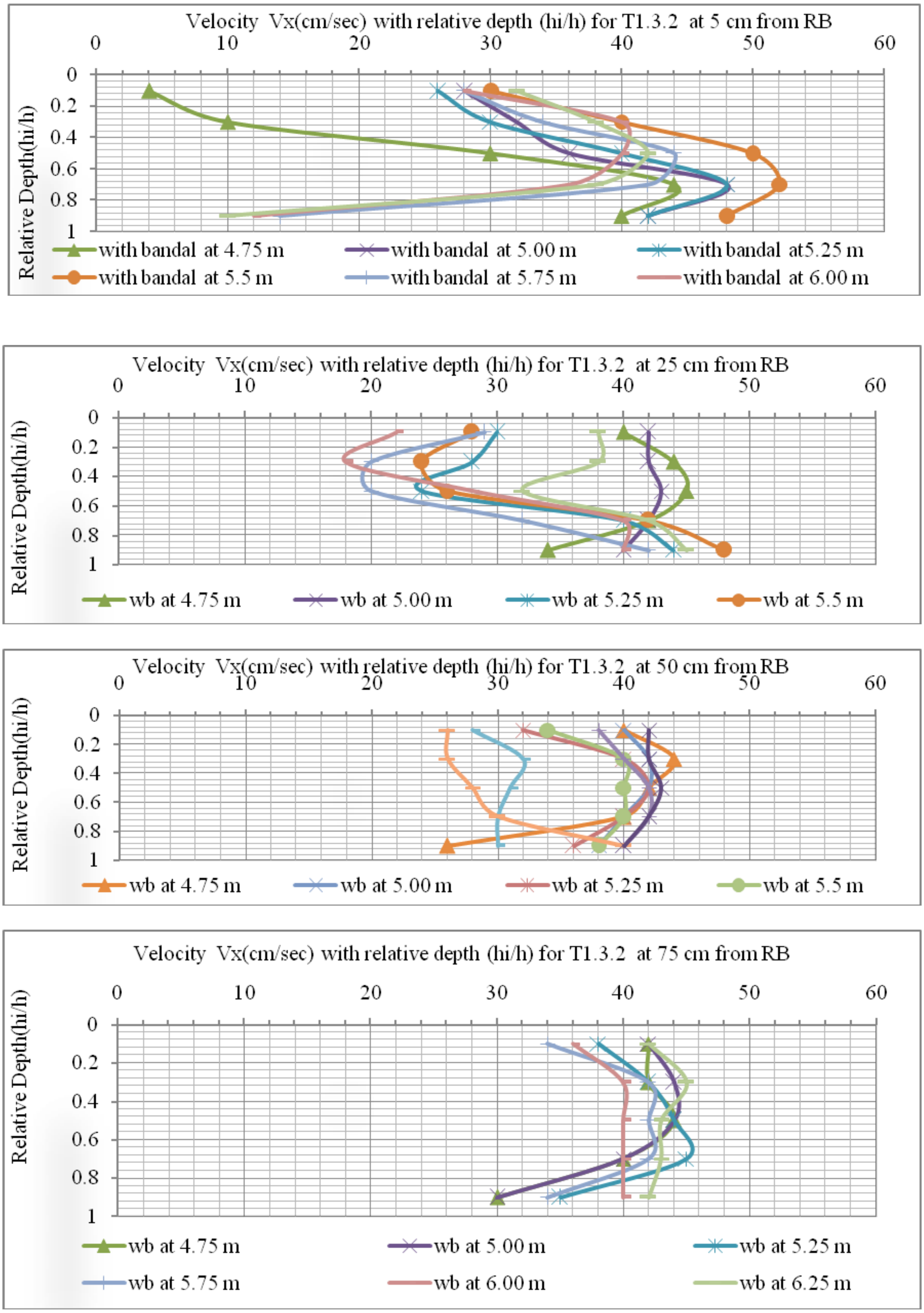

Figure 8: Velocity along the main flow of the channel for T1.3.2 at different points from the Right bank

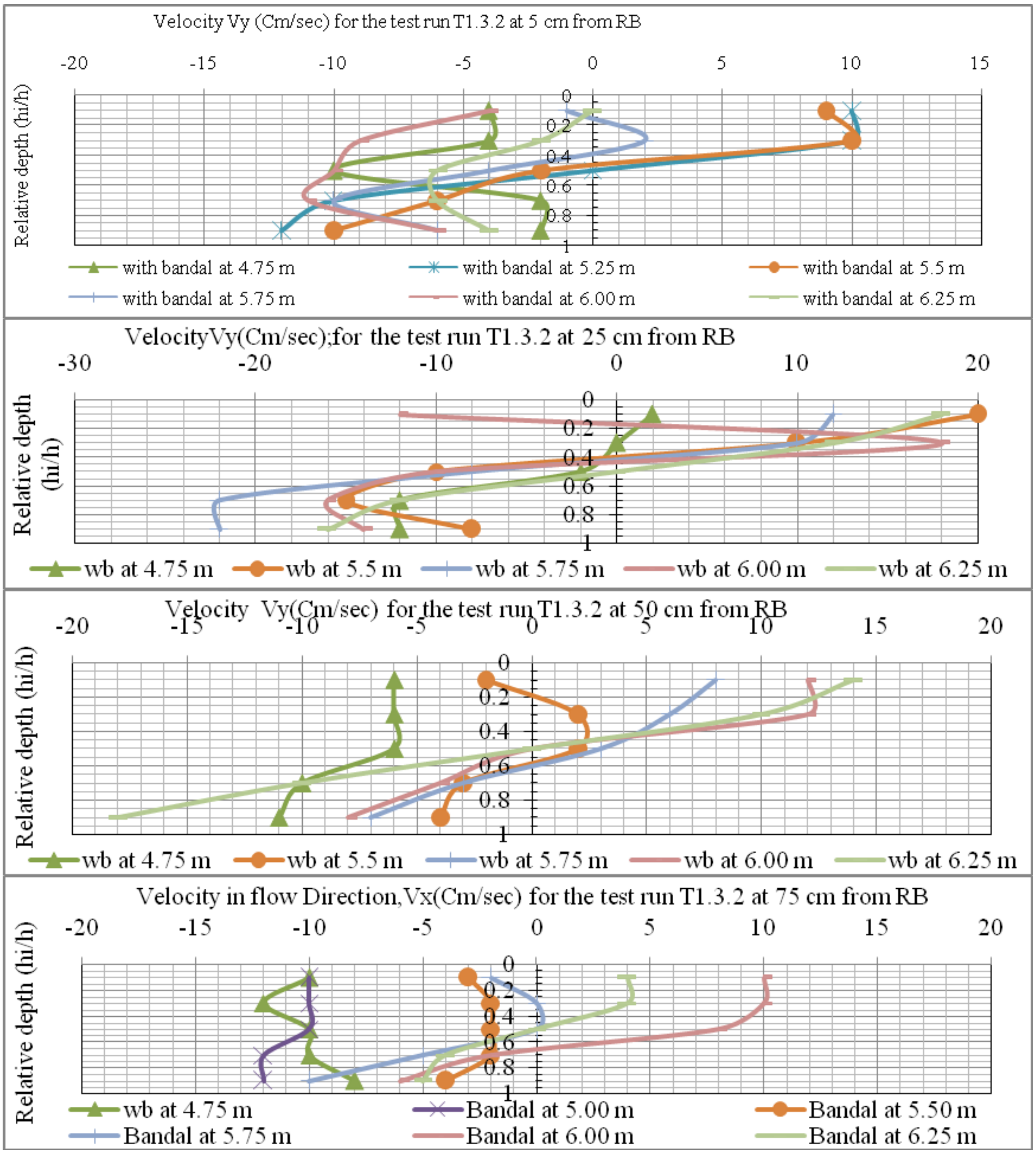

Figure 9: Velocity (Vy) perpendicular to the main flow for T1.3.2 at different points from right bank

Table 1: Position of bamboo bandals at right bank of flume the test run T1.3.2

Test No.	Bandal-1 (L=193 cm)		Bandal-2 (L=193 cm)	
	Starting of bandal (m)	Ending of bandal (m)	Starting of bandal (m)	Ending of bandal (m)
T1.3.2	4.50	6.32	11.14	12.93

where T1.3.2 is explained as T is stand for the test run, 1 is for angle at right bank with flow direction θ=20°, length of bandals L=193 cm, 3 is for spacing between bandals S=2.5L=2.5*193=483 cm

3. RESULT AND DISCUSSIONS

There are two bamboo bandalling structures are considered in the test run T1.3.2 as in Figure.9, in case of 5 cm grid line fall behind the structure very near the right bank of the flume. Vertically (along the depth of water) the location of measurement is such that at each point velocities were measured at 0.1h, 0.3h, 0.5h, 0.7h & 0.9h from the water surface where h is the depth of water at initial bed condition which is 36.00 cm. Distribution of velocities at points (4.75 m, 5.0 m, 5.25 m, 5.5 m, 5.75m, 6.0 m & so on) along the channel length at point 5 cm apart from the right bank are graphically presented for test run T1.3.2 of Figure 8. If the velocities are taken at point (4.75m, 5cm), it is seen that at 0.1h water depth velocity is much less about 5 cm/s. It increases in deeper water at points 0.3h, 0.5h & 0.7h. It appears to be realistic, because the points 0.1h, 0.3h are behind (downstream) bamboo chatai. Bamboo chatai blocked water current down to 18 cm (0.5h, 50% blockage in this test arrangement). As a result, the velocities are much less. But at 0.7h depth velocity is maximum at this grid location. It is obvious since this point is below the bamboo chatai. So the point is in the opening (water could flow freely below the chatai). Now the lowest point (at 0.9h depth) shows again relatively less velocity than that at 0.7h. Again it is realistic since it is near the bed & influenced by bed friction or other parameters.

If compared velocity graph at grid (4.75m, 5cm) with that at grid (5m, 5cm), the later graph shows higher velocities (but nature of vertical distribution is similar). It is realistic/ natural since the grid (5m, 5cm) lies downstream and further away from the structure (Chatai), the chatai has less influence on this grid point. Velocities at grid points further downstream along 5cm distance from the right bank shows velocities of higher magnitude. It is obvious, since they are away from the structure. So the structure has less influence on them.

3.1 Interpretation of main flow velocity Vx at 25cm from right bank along the laboratory channel for the test run T1.3.2

There are two bamboo bandalling structures are considered in the test run T1.3.2 as in Figure 7, the grid points at which the velocities around the Bandal-1 for 25 cm grid line 4.75m & 5.0m points velocity graph fall upstream of the structure, 5.25m & 5.5m points velocity graph is near the bandal structure &5.75m, 6.0m & 6.25m points velocity graph is downstream or behind the structure near the right bank of the flume.. Vertically (along the depth of water) the location of measurement is such that at each point velocities were measured at 0.1h, 0.3h, 0.5h, 0.7h & 0.9h from the water surface where h is the depth of water at initial bed condition which is 36.00 cm. Bamboo chatai blocked water current down to 18 cm (0.5h, 50% blockage in this test arrangement).Distribution of velocities at points (4.75 m, 5.0 m, 5.25 m, 5.5 m, 5.75m, 6.0 m & so on) along the channel length at point 25 cm apart from the right bank are graphically presented for test run T1.3.2 of Figure 8. If the velocities are taken at point (4.75m, 25cm) & (5.0m,25cm) upstream of the structures, it is seen that at 0.1h water depth velocity is about 40 cm/s. It increases in deeper water at points 0.3h & 0.7h is about 45 cm/sec and finally at 0.7h & 0.9h is reduces about 30 cm/s which follow the normal velocity profile.

If the velocities are taken at point (5.25m, 25cm) & (5.50m,25cm) just immediate upstream of the structure or at the structure, it is seen that at 0.1h water depth velocity is about 30 cm/s. It decreases in deeper water at points 0.3h & 0.5h is about 25 cm/sec and finally increases upto 45 to 50 cm/sec at 0.7h & 0.9h which does not follow the normal velocity profile. It appears to be realistic so that at depth 0.1h depth water is obstructed by the bamboo chatai so that there is less velocity of 30 cm/s where as finding no alternative velocity reduces upto 0.3h to 0.5h at about 25cm/s. After 0.5h depth, there is opening & flow velocity accumulation is occurred. In this zone water can freely pass below the bamboo chatai & velocity increased upto 45 to 50 cm/sec.

If the velocities are taken at point (5.75m, 25cm), (6.00m,25cm) & (6.25m, 25 cm) downstream of the structure or behind the structure, it is seen that at 0.1h water depth velocity is less & increases deeper water at points 0.3h & 0.5h and finally more increases 0.7h & 0.9h which does not follow the normal velocity profile.

At point (5.75m, 25 cm) velocity at 0.1h depth is 30 cm/s which decreases upto 03.h & 0.5h at 20 cm/s and from it velocity increases at 42 cm/s. It appears to be realistic so that at depth 0.1h depth water is obstructed by behind the bamboo chatai so that there is less velocity of 30 cm/s where as finding no alternative velocity reduces upto 0.3h to 0.5h at about 20cm/s. After 0.5h depth, there is opening & flow velocity accumulation is occurred. In this zone water can freely pass below the bamboo chatai & velocity increased upto 42 cm/sec.
At point (6.00m, 25 cm) velocity at 0.1h depth is 22 cm/s which decreases at 0.3h is 18 cm/s and from it velocity increases at 44 cm/s. It appears to be realistic so that at depth 0.1h depth water is obstructed by behind the bamboo chatai so that there is less velocity of 22 cm/s where as finding no alternative velocity reduces at 0.3h is 18 cm/s. After 0.3h depth there is less structural influence of point (6.0m, 25cm) from (5.75m, 25 cm), so the velocity is going to increase due to opening & flow velocity accumulation is occurred. In this zone water can freely pass below the bamboo chatai & velocity increased upto 48 cm/sec.

At point (6.25m, 25 cm) velocity at 0.1h depth is 38 cm/s which decreases at 0.3h & 0.5h is 32 cm/s and from it velocity increases at 45 cm/s. It appears to be realistic so that at depth 0.1h depth water is obstructed by behind the bamboo chatai so that there is less velocity of 38 cm/s which is greater than the points (5.75m, 25cm) & (6.om,25cm) due to relatively less structural influence. So the velocity is going to increase due to opening & flow velocity accumulation is occurred. In this zone water can freely pass below & behind the bamboo chatai & velocity increased upto 45 cm/sec.

As a result, the velocities are much higher at the structure and downstream of the structure. It is obvious from these higher velocities; the sediment is carried to the channel bank behind the structures. From the above velocity interpretation it is clear that the velocity behind the bamboo bandalling structures in the grid line 25 cm along the generate sediment & in further downstream with low velocity, there is a tendency to release the sediment there & ultimately right bank sedimentation is appeared after the test run.

3.2 Interpretation of main flow velocity Vx at 75cm to 215 cm from right bank along the laboratory channel for the test run T1.3.2

In case of 75cm to 215 cm grid line all the points (4.75m, 5.0m, 5.25m, 5.50m, 5.75m. 6.00m & 6.25m) velocities graph fall upstream of the structure as well as further away from the structure or right bank of the flume. Vertically (along the depth of water) the location of measurement is such that at each point velocities were measured at 0.1h, 0.3h, 0.5h, 0.7h & 0.9h from the water surface where h is the depth of water at initial bed condition which is 36.00 cm. Relation of measurement points vertically with structures. Bamboo chatai blocked water current down to 18 cm (0.5h, 50% blockage in this test arrangement).

4. CONCLUSIONS

From the above result & discussion, it can be concluded that there is sedimentation behind the bamboo bandalling structures due to less velocity.

5. ACKNOWLEDGEMENTS

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6. REFERENCES

Rahman, M.M., Nakagawa, H., Ishigaki, T. and Khaleduzzaman, ATM. 2003. Channel stabilization using bandalling, Annuals of Disaster Prevention Research Institute, Kyoto University, No. 46 B, pp. 613-618.

Rahman M. M., Nakagawa H. and Haque M. A.: Scouring around spur-dikes in alluvial floodplain rivers, Annual J. of Hydraulic Engineering, JSCE, Vol. 48, pp. 1075-1080

STUDY ON EXISTING DRINKING WATER SOURCES AND THE POTENTIALITY OF RAIN WATER HARVESTING AT MONGLA, BAGERHAT

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ABSTRACT

This paper deals with the finding of the existing drinking water sources and evaluation of the potentiality of rainwater harvesting at Mongla, the coastal belt of Bangladesh. From the field investigation, ponds and rainwater were found as the existing sources. Tubewell and p.s.f were found abandoned in the study area. From the laboratory test, pond water was found highly contaminated with total coliform (TC) and partially contaminated with E.coli and other parameters. The stored rain water exceeded the TC and E.coli limit. Using clay pot ceramic filter, TC and E.coli were removed by 99.45% and 100% respectively. So the potentiality of RWH can be feasible at the study area by combining clay pot ceramic filter with the rainwater tank. It is expected that the results will be highly useful in the planning and design of the project providing safe drinking water.

Keywords: *Potentiality, coastal, rainwater harvesting, contaminated, coliform and ceramic filter.*

1. INTRODUCTION

Availability of safe drinking water is one of the basic minimum requirements for healthy living. The world population is increasing day by day. As a result of rapid population growth, combined with industrialization, urbanization, agricultural intensification and water-intensive lifestyles is resulting in a global water crisis. The world supply of freshwater is not being increased. More and more people are becoming dependent on limited supplies of freshwater that are becoming more polluted. Water security, like food security, is becoming a major national and regional priority in many areas of the world.

Like many other developing countries, public water supply in Bangladesh evidently provides a shortfall in demand. The southwest coastal region of Bangladesh is the home of large numbers of poor, small and marginal farm families and shrimp workers. The period between November and May is considered as dry season. During this season only 22% of the total annual rainfall takes place in the country and the evaporation rate is four times higher than the amount of rainfall (BBS, 2001). This result is a scarcity of water because of the decline of water flow in the rivers and the drying up of large numbers of water bodies.

The coastal belt of Bangladesh suffers from high salinity in surface and ground water, arsenic contamination of shallow aquifer, lack of aquifer and difficulties in extracting saline free water are some of causes. There are certain areas in the coastal belt of Bangladesh where tube wells are not successful, because ground water is mostly saline to depths of 700-1000 ft. (DPHE-Unicef, 1994) and suitable freshwater aquifers are not available. At least 59 districts of Bangladesh out of 64 have been reported exposed to arsenic problem (DPHE-WHO, 2000). It is high time to be concerned about the drinking water problem of coastal area like Mongla. It is high time to be concerned about the drinking water problem of coastal area like Mongla.

Rainwater harvesting, the technology for collecting and storing rainwater for human use from rooftop or any other means, may be an alternative solution to solve the present scarcity of water. This study is conducted to evaluate the existing drinking water sources and the opportunity of rainwater harvesting at Mongla upazila of Bagerhat district of Bangladesh. The main objectives of the study are evaluating the existing drinking water problems and develop corresponding models to ensure safe drinking water at the study area.

2. METHODOLOGY

This study was conducted in the Mongla upazila of Bagerhat district, Bangladesh. To complete the study, a strategy including some qualitative and quantitative methods and analysis was taken. From the upazila two unions (Chila and Chandpai union) were selected as the study area. Preliminary and detailed field investigation was done through questionary survey among some group of people to identify technical and social problems of existing drinking water sources. Water qualities of the existing drinking water sources were investigated through laboratory test. The quality parameters of stored rainwater at dry season were investigated regularly.

To remove coliforms from the stored rainwater, clay pot ceramic filter was made by ceramic filter core and clay pot (10L). The performance of clay pot ceramic filter to remove coliforms from water was investigated. After that all experimental data was analyzed and some suggestions for the potential rainwater harvesting at the study area.

3. RESULTS AND DISCUSSION

3.1 Existing Drinking Water Sources

From the field visit and questionnaire survey in the study area it was found that about 37% people use pond water and 63% use rain water. From the collected data the source of drinking water was found as follows:

Table-1: Drinking Water Sources

union	Shallow Tubewell		Deep Tubewell		P.S.F		Pond		Rainwater Harvesting	
	active	inactive	active	inactive	active	inactive	active	inactive	active	inactive
Chila	0	2	0	0	0	28	8	0	24	0
Chandpai	0	6	0	1	0	16	13	0	47	0

3.1.1 Present Condition of Pond Management System

From field investigation on pond it was observed that-

- ❖ Considerable amount of algae growth has been found in some of the pond.
- ❖ The embankments around 80% of the ponds are not well protected.
- ❖ Users are not sufficiently aware of maintenance of the pond.
- ❖ The caretaker families and the users lack in technical, operation and maintenance requirements.

3.1.2 Present Condition of PSF System

Due to lack operation and maintenance of PSF and pond, all the PSF were abandoned. By the investigation, it is found that, the PSFs were not cleaned in proper time. The most users were not aware of pure drinking water. Some PSFs had the problem of hand pump, valve and piston of the tube well. Involvement of the user group for regular monitoring, maintenance, and repair was absent also. As a result, most of the PSF became useless after six month to one year from the starting of PSF for use.

3.1.3 Present Condition of Tubewell System

Most of the tube-well water contains high quantity of iron and salinity. Some tube-well water was arsenic contaminated. Some tube-well were not at sufficient distance from latrine. Because of bad drinking test all of existing sallow and deep tubewells are not used as drinking water sources, but they are used for other purposes.

3.1.4 Present Condition of Rain Water Harvesting System

In the study area, previously, the rainwater is harvested by conventional system. It was generally collected from different types of roof and stored in motka, pitcher and dram. The collected rain water is used for different purposes. In recent few years, with the help of several NGOs, rain water harvesting in promoting in the area. Now a day, along with different conventional system, larger rainwater reserver tank is being used for the storage of rain water using for drinking purpose at dry season. But due to maintenance and operational problems, stored rain water is being contaminated.

Probable causes of contamination of rainwater during collection of the water are-

- Contamination from the catchment surface materials.
- Accumulation of dirt, leaves, excrement of birds and small animals etc.
- Contamination by human beings.

Probable causes of contamination of rainwater during storage of the water are-

- Entry and mixing with polluted water.
- Entry of dirt leaves etc.
- Algae growth.
- Corrosion of the reservoir material.
- Transmission of disease organisms by insects.

3.2 Assessment of Water Quality Parameters of Existing Sources

Samples from selected drinking water sources were collected and brought to laboratory to determine water quality parameters. From the test result the variation of different parameters was presented by figure. All the tests were performed in accordance with standard method. For laboratory investigations, it was intended to determine the relevant parameters of collected water from the existing drinking water sources of the study area as presented in Table-2 and Table-3.

Table-2: Water Quality Parameter Results of Existing Ponds

Source of water	Unit	Pond-1	Pond-2	Pond-3	Pond-4	Pond-5
pH	-	6.98	8.15	7.49	7.23	7.89
Color	Pt-co	140	43	60	111	84
Turbidity	NTU	17.2	9.4	11.5	23.9	7.79
TDS	mg/l	360	970	810	790	850
SS	mg/l	127	80	96	110	130
Hardness	mg/l	290	390	160	180	172
Alkalinity	mg/l	165	600	95	102	180
Cl^-	mg/l	237	725	452	350	440
TC	N/100ml	846	655	925	1120	970
E. coli	N/100ml	32	80	28	45	36

From the water quality parameter investigation of 5 existing ponds, shown in Table-2, it is found that, the water quality exceeds the standard limit of color, turbidity, suspended solid, TC and E. coli. The other parameter pH, TDS, hardness, alkalinity and chloride was found to be in limiting value. This indicates that proper treatment is required before using the pond water for drinking purpose.

Table-3: Water Quality Parameter Results of Existing Shallow Tubewells

Source of water	Unit	Tubewell-1	Tubewell-2	Tubewell-3	Tubewell-4	Tubewell-5
pH	-	6.99	7.61	7.34	7.82	7.39
Color	Pt-co	310	322	343	268	314
Turbidity	NTU	80.5	82.2	90.6	78.2	103.1
TDS	mg/l	1590	1320	1480	1980	1840
SS	mg/l	62	74	80	50	64
Hardness	mg/l	420	300	450	380	480
Alkalinity	mg/l	1000	780	655	820	985
Cl-	mg/l	1100	810	890	1460	1096
Fe	mg/l	5.60	2.54	4.42	1.89	1.58
As	ppb	0.5	1.5	1	2	0.5

From the water quality parameter investigation of 5 existing shallow tubewells, shown in Table-3, it is found that, the water quality exceeds the standard limit of color, turbidity, TDS, suspended solid, chloride and iron. Before using as drinking water, tubewell water needs proper treatment. Due to bad color and taste, people are unwilling to use this water for drinking purpose. As a result most of the tubewells are abandoned for drinking purpose.

3.3 Variation of Water Quality Parameters of Stored Rain Water during Dry Season

The quality parameter of stored water was investigated from March to June month to verify the quality of stored water in the rainwater storage tank after 4-5 month of the storage of water, during the dry season. Variation of different parameters are described and shown in figure below.

3.3.1 Variation of pH

Figure-1 shows the variation of pH of stored rain water of 5 different rainwater reserver tanks. The pH of stored rainwater was found to be increase in range with time.

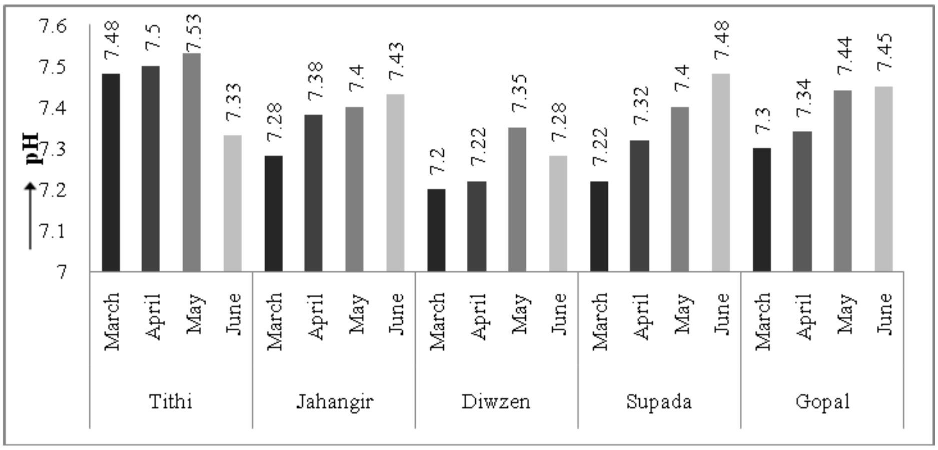

Figure-1: Variation of pH of Stored Rainwater in Dry Season

3.3.2 Variation of Turbidity

Figure-2 shows the variation of turbidity of stored rain water of 5 different rainwater reserver tanks. The turbidity of stored rainwater was found to be decrease with time.

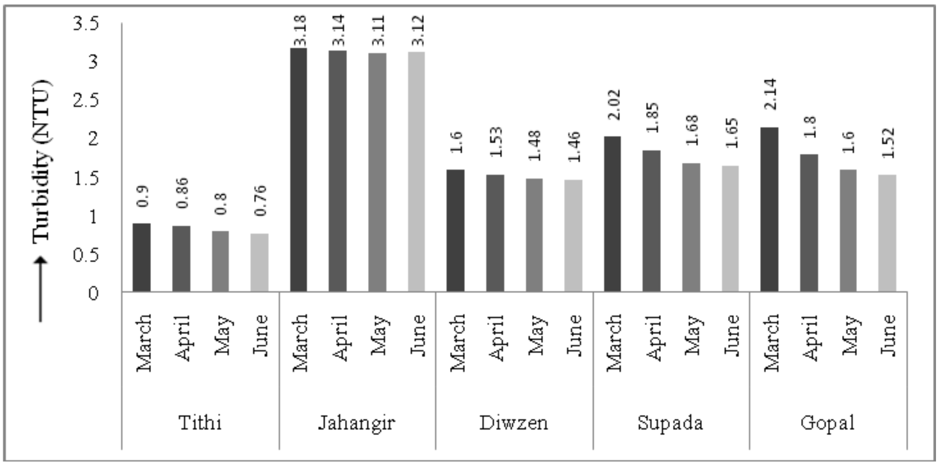

Figure-2: Variation of Turbidity of Stored Rainwater in Dry Season

3.3.3 Variation of Chloride

Figure-3 shows the variation of Chloride value of stored rain water of 5 different rainwater reserver tanks. The Cl^- value of stored rainwater was found to be increase with time.

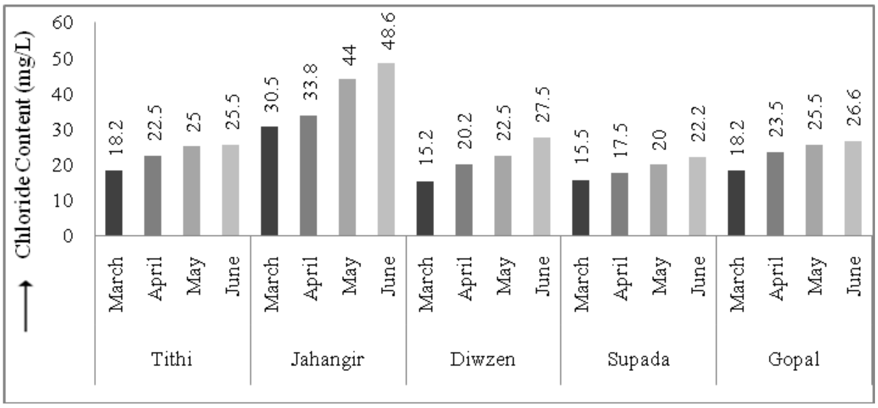

Figure-3: Variation of Chloride of Stored Rainwater in Dry Season

3.3.4 Variation of Total Coliform (TC)

Figure-4 shows the variation of TC in stored rain water of 5 different rainwater reserver tanks. The TC of stored rainwater was found to be increase with time exceeding the standard limit. This contamination can be caused due to the collection of rainwater from unclean roof, entry of leaves and organic material in the tank, entry of vermin and mosquito or any other maintenance difficulties. Since the coliforms get suitable environment in the storage tank, they increases in number.

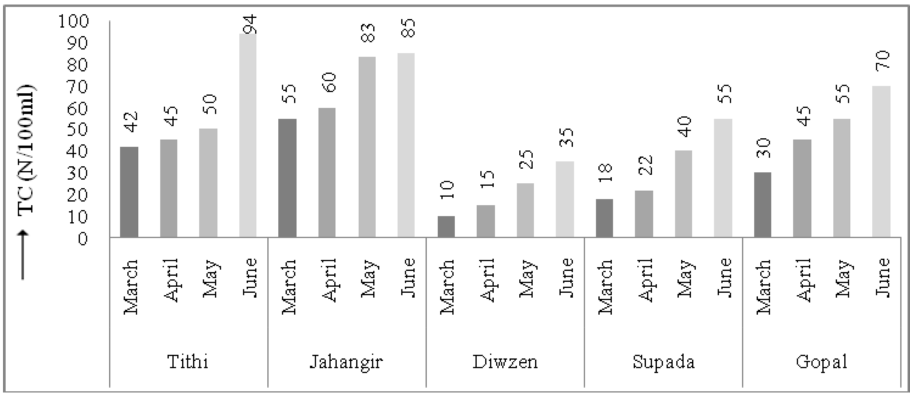

Figure-4: Variation of Total Coliform (TC) of Stored Rainwater in Dry Season

3.3.5 Variation of E.coli

Figure-5 shows the variation of E.coli in stored rain water of 5 different rainwater reserver tanks. The E.coli of stored rainwater was also found to be increase with time and exceed the standard limit.

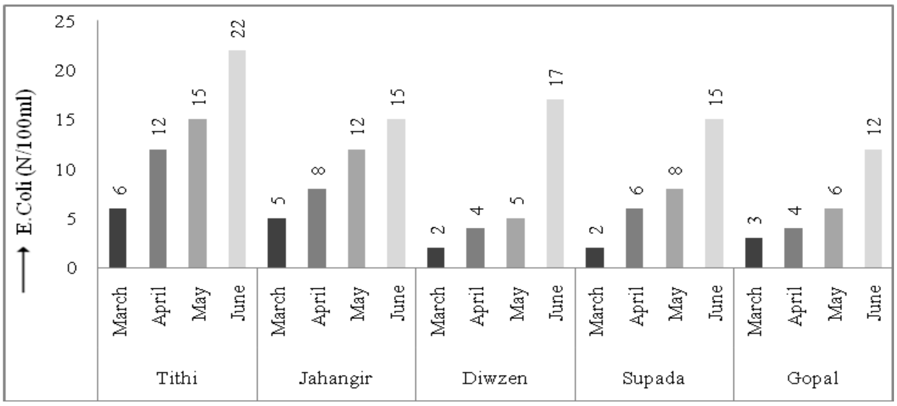

Figure-5: Variation of E.Coli of Stored Rainwater in Dry Season

From the investigation it is found that, most of the parameters of stored rainwater are in the limiting value, without total coliform (TC) and E.coli. For this reason, we used a low cost ceramic filter (clay pot) to remove this TC and E.coli.

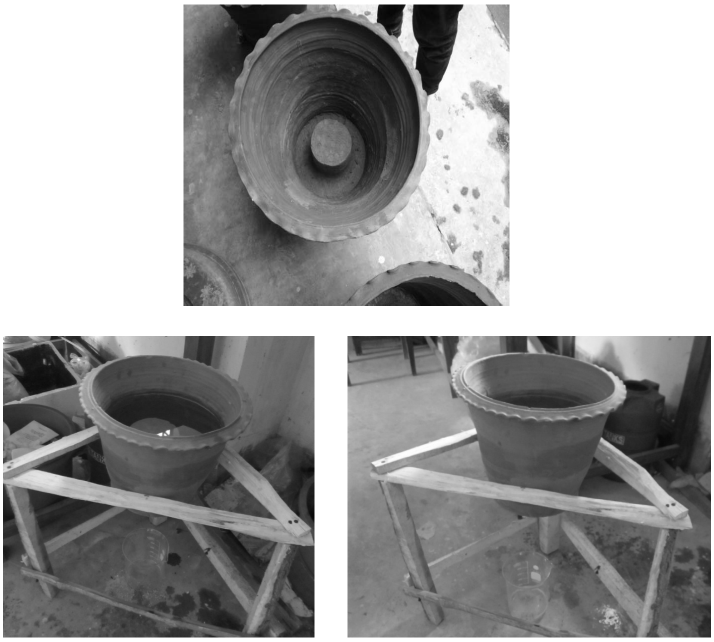

Figure-5: Simple Low Cost Clay Pot Ceramic Filter

3.4 Performance of Ceramic Filter

To investigate the performance of the ceramic filter, 5 samples were taken. Analyzing the TC and E.coli removal efficiency was found to be 99.45% for TC and 100% for E.coli. The filtration rate or discharge rate was also measured and found 2 lit/hr.

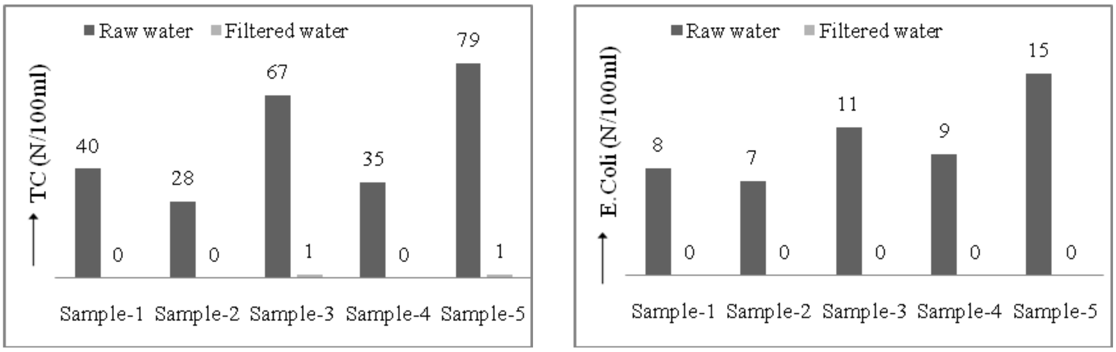

Figure-7: Removal Efficiency of Ceramic Filter for E.coli

4. CONCLUSIONS

After detailed field investigations and laboratory investigations it was observed that the main sources of drinking water were pond and rain water in the study area. The directly used pond water is a considerable health risk to the consumer due to high contamination. From the questionnaire survey report it was also clear that people were known about the quality of rainwater and they were willing to pay to install this system as because of their limited source of safe water. So, it is clear that this system is socially feasible. Though rain water harvesting is being promoted in the area with the help of different NGOs, the quality of stored rainwater exceeds the TC and E.coli limit due to different collection and maintenance difficulties. So, a low cost clay pot ceramic filter is being suggested combining with the storage tank to ensure the potentiality of RWH at the study area providing safe drinking water to the consumers.

REFERENCES

Ahmed M.F & Rahman (2000), Water Supply and Sanitation, Rural and Low Income Urban Communities, ITN-Bangladesh, BUET, Dhaka.

Ahmed M.F and Jahan H. (2000), Participatory Management of Low Cost Water Supply and Sanitation, ITN-Bangladesh, BUET, Dhaka.

APHA, AWWA & WPCF (1998), Standard methods for the Examination of water and waste water, 19th ed. Bangladesh Bureau of Statistics, (1997) Statistical Year Book of Bangladesh, Bureau of Statistics, Ministry of Planning, Gov. of peoples Republic of Bangladesh.

Bangladesh Bureau of Statistics (2001), Statistical Year Book of Bangladesh, Bureau of Statistics, Ministry of Planning, Govt. of peoples Republic of Bangladesh.

BGS and DPHE (2000), Arsenic contamination of groundwater in Bangladesh, Kinniburgh, D. G. and Smedley, P. L. (Editors),vol.2, Final Report, BGS Technical Report WC/00/19, British Geological Survey, Keyworth, United Kingdom.

BGS and DPHE (2001), Arsenic contamination of groundwater in Bangladesh. BGS Technical Report, WC/00/19.

DPHE-UNICEF (1994), Study to Forecast Declining Ground Water Level in Bangladesh, June 19.

DPHE/BGS/MML (1999), Groundwater Studies for Arsenic Contamination in Bangladesh. Report to Government of Bangladesh and DFID, British Geological Survey and Mott MacDonald.

DPHE-UNICEF (1994), Study to Forecast Declining Ground Water Level in Bangladesh, June 19.

Dhaka Community Hospital (DCH) (2000), Arsenic in Bangladesh: Report on the 500- Village Rapid Assessment Project, Dhaka, Bangladesh: 1--48.94.

MPO (1991). Groundwater Resources of Bangladesh, Technical Report No. 5, Master Plan Organization, Ministry of Water Resources, Government of Bangladesh. Harza Engineering USA in association with Sir MacDonald and Partners, UK, Met Consultant, USA and EPC Ltd, Dhaka.

IRC (1983) Small Community Water Supplies, 2nd edition, IRC Technical Paper No.18, IRC International Water and Sanitation Centre, The Hague.

IRC (1991) Partners for Progress: An approach to sustainable piped water supplies, IRC Technical Paper No.28, IRC International Water and Sanitation Centre, The Hague.

SOES-DCH, (2000), "Groundwater Arsenic Contamination in Bangladesh, A Summary of 239 days field survey from August 1995 to February 2000, B. Twenty-seven days detailed field survey information from April 1999 to February 2000, School of Environmental Studies, Jadavpur University, Calcutta 700 032, India and Dhaka Community Hospital, Dhaka 1217, Bangladesh.

World Health Organization (WHO), (1993), Guidelines for Drinking Water Quality, 2nd edition, Volume 2.

World Health Organization, (2011), Guidelines for Drinking-water Quality, 4th edition. Water Aid (1996a) Children and Water, Development Issue, Water Aid, London.

Water Aid (1996), Women and Water, Development Issue, Water Aid, London.

Water Aid (1997), Water Aid Annual Review, Water Aid, London.

APPENDIX

Table A: Variation of Quality Parameter of Stored Rain Water in Tithi's Rain Water Reserver Tank (Gazi Tank)

month		March	April	May	June
pH	-	7.48	7.5	7.53	7.33
Color	Pt-co	0	0	0	0
Turbidity	NTU	0.9	0.86	0.8	0.76
Alkalinity	mg/l	25	28	30	32
Hardness	mg/l	14.8	12.5	12.2	10.6
Chloride	mg/l	18.2	22.5	25	25.5
TC	N/100ml	42	45	50	94
EC	N/100ml	6	12	15	22

Table B: Variation of Quality Parameter of Stored Rain Water in Jahangir's Rain Water Reserver Tank (Large R.C.C. Tank)

month		March	April	May	June
pH	-	7.28	7.38	7.4	7.43
Color	Pt-co	0	0	0	0
Turbidity	NTU	3.18	3.14	3.11	3.12
Alkalinity	mg/l	72	78	84	92
Hardness	mg/l	13.9	13.6	12.9	13.5
Chloride	mg/l	30.5	33.8	44	48.6
TC	N/100ml	55	60	83	85
EC	N/100ml	5	8	12	15

Table C: Variation of Quality Parameter of Stored Rain Water in Dwijen's Rain Water Reserver Tank (Small R.C.C Tank)

month		March	April	May	June
pH	-	7.2	7.22	7.35	7.28
Color	Pt-co	0	0	0	0
Turbidity	NTU	1.6	1.53	1.48	1.46
Alkalinity	mg/l	80	82	85	90
Hardness	mg/l	13.9	13.6	13.5	14.3
Chloride	mg/l	15.2	20.2	22.5	27.5
TC	N/100ml	10	15	25	35
EC	N/100ml	2	4	5	17

Table D: Variation of Quality Parameter of Stored Rain Water in Supada's Rain Water Reserver Tank (Large R.C.C. Tank)

month		March	April	May	June
pH	-	7.22	7.32	7.54	7.88
Color	Pt-co	0	0	0	0
Turbidity	NTU	2.02	1.85	1.68	1.65
Alkalinity	mg/l	65	68	72	75
Hardness	mg/l	9	9.6	10.3	12.5
Chloride	mg/l	15.5	17.5	20	22.2
TC	N/100ml	18	22	40	55
EC	N/100ml	2	6	8	15

Table E: Variation of Quality Parameter of Stored Rain Water in Gopal's Rain Water Reserver Tank (Gazi Tank)

month		March	April	May	June
pH	-	7.3	7.34	7.44	7.48
Color	Pt-co	0	0	0	0
Turbidity	NTU	2.14	1.8	1.6	1.52
Alkalinity	mg/l	30	33	41	44
Hardness	mg/l	8.95	8.5	8.2	8.55
Chloride	mg/l	18.2	23.5	25.5	26.6
TC	N/100ml	30	45	55	70
EC	N/100ml	3	4	6	12

Table F: Removal Efficiency of Ceramic Filter for TC

	Total Coliform (TC) N/100ml	
Sample	Raw water	Filtered water
Sample-1	40	0
Sample-2	28	0
Sample-3	67	1
Sample-4	35	0
Sample-5	79	1

Table G: Removal Efficiency of Ceramic Filter for E.coli

	E. Coli N/100ml	
Sample	Raw water	Filtered water
Sample-1	8	0
Sample-2	7	0
Sample-3	11	0
Sample-4	9	0
Sample-5	15	0

SCOURING IN JAMUNA RIVER AND FAILURE OF SIRAJGONJ HARDPOINT

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ABSTRACT

Bed level variation is one of the major causes of river bank erosion. The Jamuna is a braided river with bank materials that are highly susceptible to erosion. It is observed that, around 3.5 km long Sirajganj hard point (revetment) along the right bank of the Jamuna have been facing scour related damages since its construction. In this study, the scouring of river bed of Jamuna river and damaging of hardpont at Sirajgonj point is analyzed based on secondary bathymetry survey data collected from Bangladesh Water Development Board (BWDB) for the years 2007 to 2012. It is observed that, variation of bed level at the straight portion of revetment is higher compared to upstream portion. The yearly maximum scour depth was found to be varied from 10 m to 25 m that occurs during the month of July It is found that, failure of hardpoint was occurred for the maximum scour depth ranged from 18 m to 25 m. It is observed that, there is a good relation between failures of hardpoint with maximum scour depth. The design of the revetment wall is checked using Ingis equation of Lacey's regime depth, and it is observed that the provided depth of revetment wall is much lower than the estimated requied depth.

Keywords: *Hardpoint, Scour depth, Jamuna river, Bed level variation, River bank erosion*

1. INTRODUCTION

Bed level variation is one of the major causes of river bank erosion. Annually thousands of hectares of land are eroded along the major rivers. Millions of people became homeless and landless due to erosion. The erosion also affects in the morphological behaviours of the rivers. The radical morphological change in a sand-bed braided river is a great challenge for the structural stability of the river training works. The Jamuna is a braided river with bank materials that are highly susceptible to erosion. There are insufficient evidences on field-based research in a large scale sand bed braided river. Especially in the river where frequent failure event occurs of the river training works due to rapid morphological change. To protect bank erosion, more than thirty numbers bank protection structures (hardpoint, groin and RCC spur) have been constructed along both banks of the Jamuna River. Almost every year a number of structures encounter different types of failure problems and maintenance of such failure or damaged events are one of the big challenges to Bangladesh Water Development Board (BWDB). Rahman et al. (2012a,b) reported that around 3.5 km long Sirajganj hard point (revetment) along the right bank of the Jamuna have been facing scour related damages since its construction in 1998. Historically, the damaged portion is not concentrated near the upstream termination (typical); rather it is related with the position, orientation and alignment of adjacent sand bars that guide flow towards the bank line anywhere of the structure. River training is being practiced in Bangladesh from 1960s. Bandalling is practiced for navigation where as revetment works, hardpoint, groynes/spur, guide bank, closing of secondary channels are practiced for bank protection. Most of the erosion protection structures were either solid spurs or revetment to protect important locations like Kamarjani, Fulchari, Sariakandi, Kazipur and Sirajgonj in the Jamuna River. In case of the Ganges River, protection structures have been constructed in many places like Panka-Narayanpur, Godagari, Rajshahi, Sarda, Pakshey, Pabna etc. along the left bank. Hard point is very massive structure. The hard point including 2.5 km revetment at Sirajgonj, revetment with concrete block at Titporal, revetment with geo-bag at Nakalia-Bera, revetment concrete block at Chandpur and RCC Spur at Panka-Narayanpurare the defensive structures and revetment with concrete block at Rajshahi, RCC Spur at Betil-Enayetpur and bandals are the offensive structures. The main purpose of these structures is to reduce the flow velocity and minimize the river bank erosion. Hard points are isolated bank revetment works with upstream and downstream termination. The primary purpose of hardpoint is to channelize the flow. However, hard points are also designed to locally protect strategically important areas, such as bridges, large settlements and ferry terminal. The hard point study is very useful to realize the actual problem near bank protection works and bed level variation and what measures should be taken to mitigate the problem. River training by hard points consists of series of short spurs constructed of stones or other erosion resistant materials, extending towards a river from the bank and having a root of stone in a trench ending 10-15 m into the bank. This so-called hardpoint system is said to be effective along straight or relatively flat convex banks, where stream lines are more or less parallel to the bank. Hard points should be used for shallow depths (depth less than 3 m within 15 m from the bank) only.

Hard points were constructed at an angle of 10^0-20^0 downstream and spaced about 75 m apart, allowing limited erosion to continue along the bank between structures. During construction of Jamuna Multi-purpose Bridge, the authority decided to construct two hardpoints at alternate banks (Bhoapur and Sirajgonj) in the Brahmaputra river to protect the guide banks of the Jamuna Bridge. At that time, the Sirajgonj town protecting embankment was declared as Sirajgonj hardpoint. Fig. 1 shows Sirajgonj hardpoint.

Figure 1: Sirajgonj hardpoint

At present, Sirajgonj hard point (a revetment-like structure) is one of the important river training works along the right bank of the sand bed braided Jamuna river to protect the Sirajgonj town. Since its construction in 1998, the hardpoint has been damaged several times and the damaged part of the structure not necessarily concentrated near the upstream termination (expected point of maximum local scour), rather straight parts of the hard point are also vulnerable to damage (Uddin, 2009). Fig. 2 and Fig. 3 shows damaged portions of Sirajgonj hardpoint in 10 July, 2009 and in 21 July,2011 respectively.

Figure 2: Damaged portion in 10 July, 2009

Figure 3: Damaged portion in 21 July, 2011

2. STUDY AREA AND METHODOLGY

2.1 Study Area

The Sirajgong is an old established town in Bangladesh. The urban and peri-urban development has been expanded close to braided Jamuna river bank. Braided rivers are characterized by having a number of alluvial channels with bars and islands in a sequence of confluence and bifurcation. The braided river transports huge volume of sediment load. The channel shifts it courses due to deposition of the excess sediment load and rapid bank erosion. Flow divergence is associated with flow deceleration and sediment deposition. The divergent flow may impinge on the bank at an increased angle leading to bank erosion, channel widening and local increased in sediment load. Fig. 4 shows the map of Sirajganj Sadar Upazila.

Figure 4: Map of Sirajganj Sadar Upazila

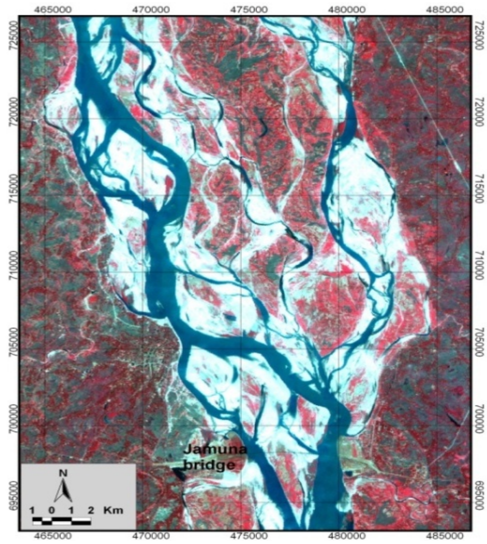

Figure 5: Satellite Image of Jamuna river

The Jamuna River is the lowest reach of the Brahmaputra River in Bangladesh. It drains an area of 550,000 km^2, and the mean annual discharge is 20,000 m^3/s. It is a large braided sand-bed river, and the number of braids at low flows varies between 2 and 3. The total width of the braided channel pattern varies between 5 and 17 km. It is a wandering braided river with an average bank full width of about 11 km with a bank full discharge of approximately 48,000 m^3/sec (Julien, 2002). The channel has been widening, increasing from an average of 6.2 km in 1834 to 10.6 km in 1992. At the confluence with the Ganges, the average annual flood is 60,000 m^3/s and low-flow discharges vary between 4,000 and 12,000 m^3/s. The maximum discharge recorded in 1988 reached 100,000 m^3/s. Klaassen (1993) reported that the Jamuna River is quite active, with frequent channel shifts and lateral migration rates frequently exceeding 500 m/yr. The shifting rate of the first-order channel is 75 to 150 m. Bank erosion rates of second-order channels of 250 to 300 m are common. Fig. 5 shows the satellite image of Jamuna river.

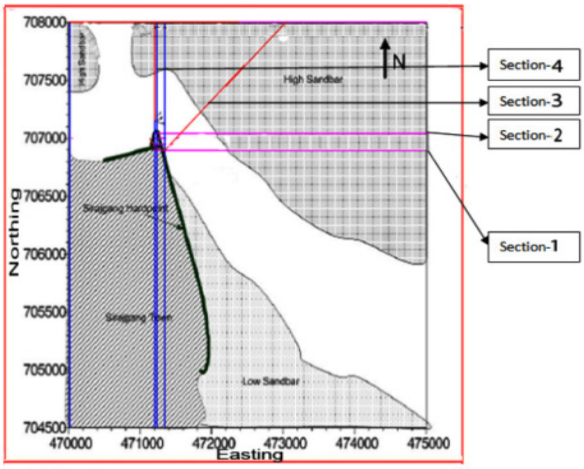

Figure 6: Plan of Sirajgonj hardpoint showing different sections. (Source: BWDB)

2.2 Methodolgy

Secondary bathymetry survey data of Jamuna river near Sirajgonj hardpoint collected from Bangladesh Water Development Board (BWDB) for the years 2007 to 2012 has been used to calculate the bed level variation and scour depth. The monthly variation and seasonal variations of bed level were calculated for four sections around Sirajgonj hardpoint and variations are compared for different sections. Maximum scour depth for different years is calculated to find the correlation between failure histories of hardpoint with scour depth.

Figure 6 **shows p**lan of Sirajgonj hardpoint indicating different sections for calculating bed level variation **and scour depth**. The coordinates of the sections are given below.

Section-1: Northing constant at 706900mN, Easting varied from 471350mE.
Section-2: Northing constant at 707050mN, Easting varied from 471350mE.
Section-3: Northing varied from 707180mN, Easting constant at 471243mE.
Section-4: Northing varied from 707170mN, Easting constant at 471208mE.
Fig. 7 shows secondary bathymetry survey data of Sirajgonj Hardpoint on 2 January, 2008, which has been collected from Bangladesh Water Development Board (BWDB).

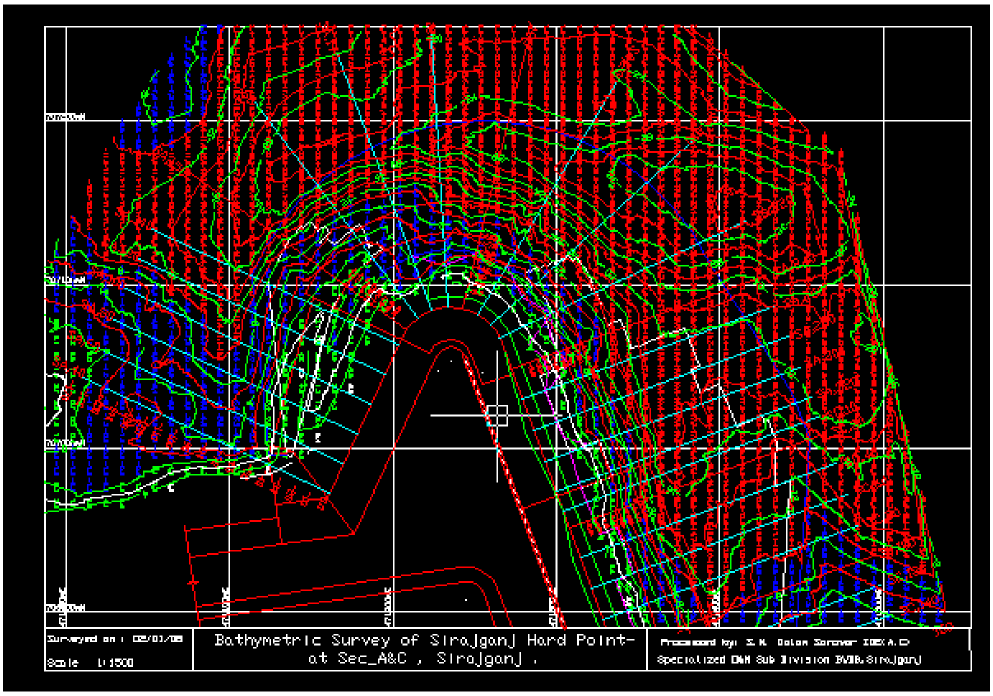

Figure 7: Sample bathymetry survey data of Sirajgonj Hardpoint on 2 January, 2008 (Source: BWDB)

3. RESULTS AND DISCUSSIONS

3.1 Monthly Variation of Bed Level

Fig. 8 shows monthly variation of bed level at section 1 in 2007. Here, northing is constant at 706900mN and easting varied from 471350mE. Large variation of bed level is observed for different months at section 1 in 2007. The maximum variation of scour depth is found 9 m (30 ft) among different months. Fig. 9 shows monthly variation of bed level at section 3 in 2007. Here, easting is constant at 471243mE and northing varied from 707180mN. No bed level variation is observed on 6th August and 6th September. The maximum variation of scour depth is found 7 m (22 ft) among different months. Fig. 10 shows monthly variation of bed level at section 4 in 2007. Here, easting is constant at 471200mE and northing varied from 707000mN. Slight variation of bed level is observed for different months at section 4 in 2007. The maximum variation of scour depth is found 6 m (20 ft) among different months. Fig. 11 shows monthly variation of bed level at section 1 in 2009. Large bed level variation is observed for different months at section 1 in 2009. The maximum variation of scour depth is found to be 13 m (43 ft) among different months. Fig. 12 shows monthly variation of bed level at section 3 in 2009. Bed level is gradually varied for different months at section 3 in 2009. The maximum variation of scour depth is found to be 10 m (33 ft) among different months. Fig. 13 shows monthly variation of bed level at section 4 in 2009. Small variation of bed level is observed for different months at section 4 in 2009. The maximum variation of scour depth is found to be 5 m (17 ft) among different months.

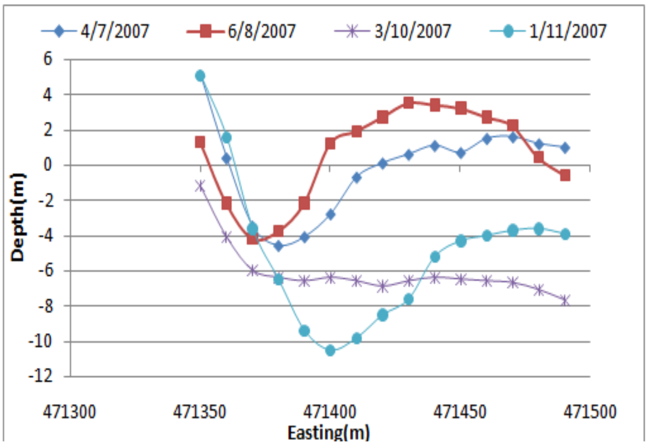

Figure 8:Monthly variation of bed level at section 1 in 2007.

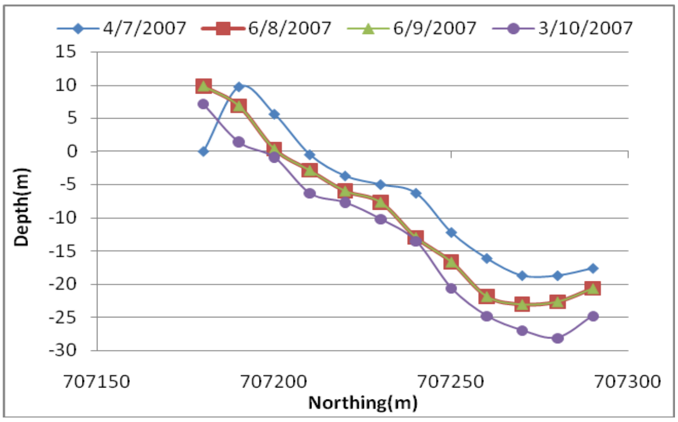

Figure 9: Monthly variation of bed level at section 3 in 2007.

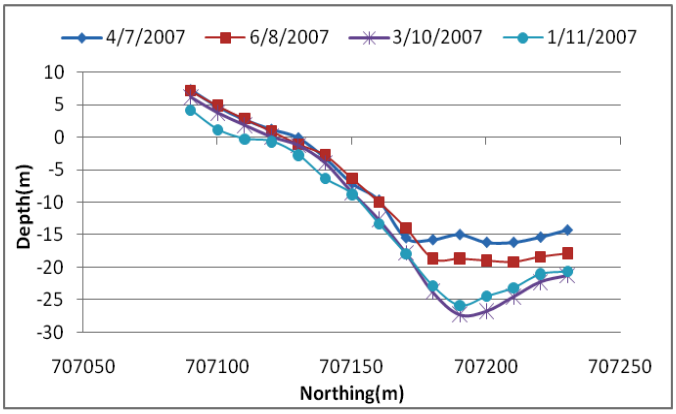

Figure 10: Monthly variation of bed level at section 4 in 2007.

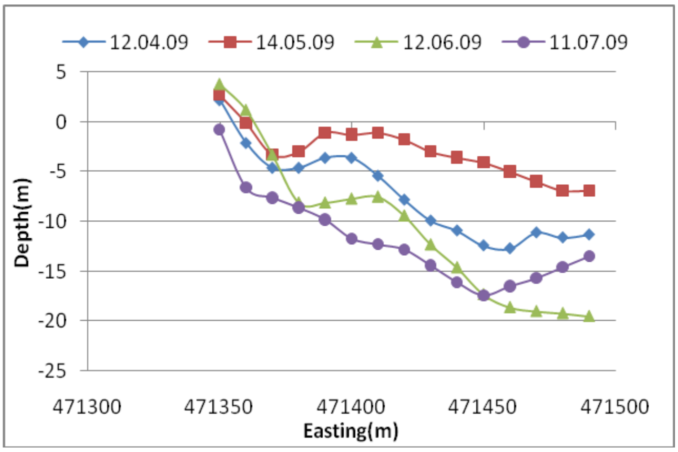

Figure 11: Monthly variation of bed level at section 1 in 2009.

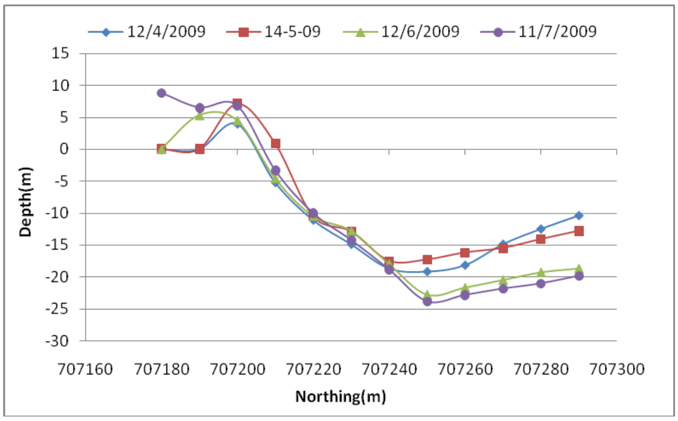

Figure 12: Monthly variation of bed level at section 3 in 2009.

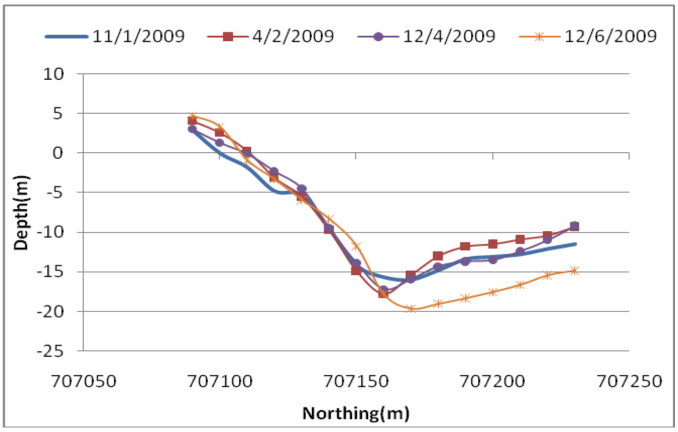

Figure 13: Monthly variation of bed level at section 4 in 2009.

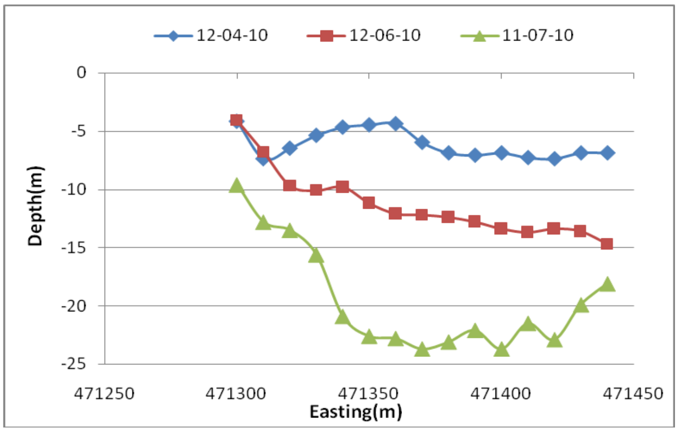

Figure 14: Monthly variation of bed level at section 1 in 2010.

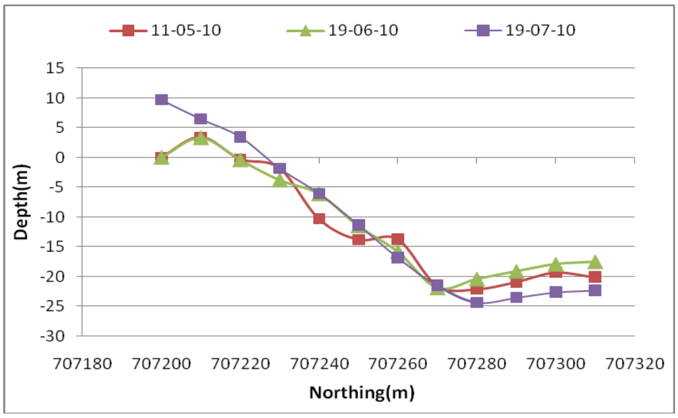

Figure 15: Monthly variation of bed level at section 3 in 2010.

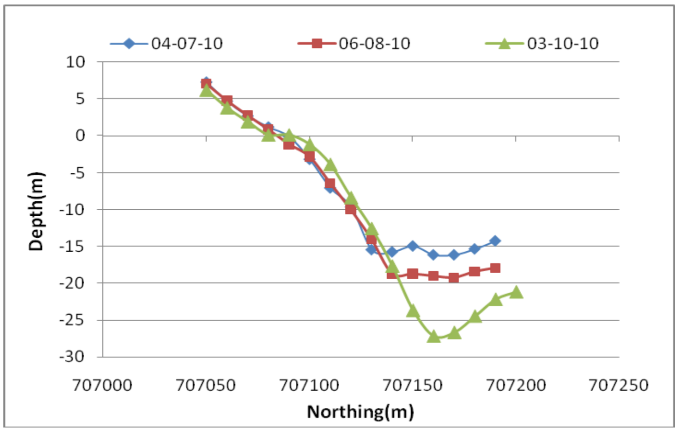

Figure 16: Monthly variation of bed level at section 4 in 2010.

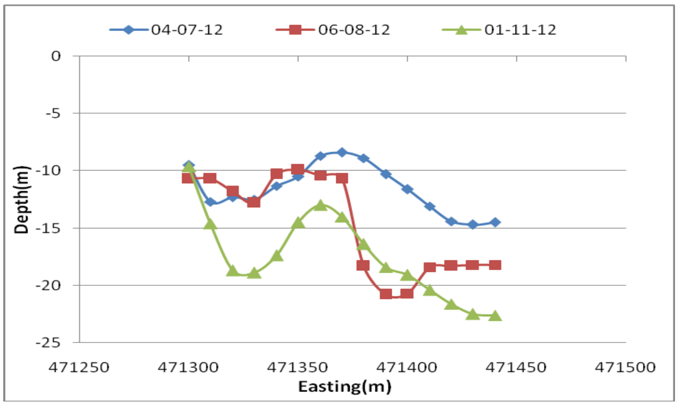

Figure 17: Monthly variation of bed level at section 1 in 2012.

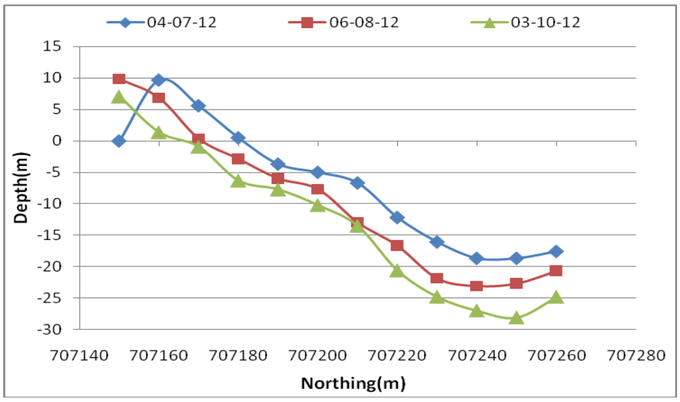

Figure 18: Monthly variation of bed level at section 3 in 2012.

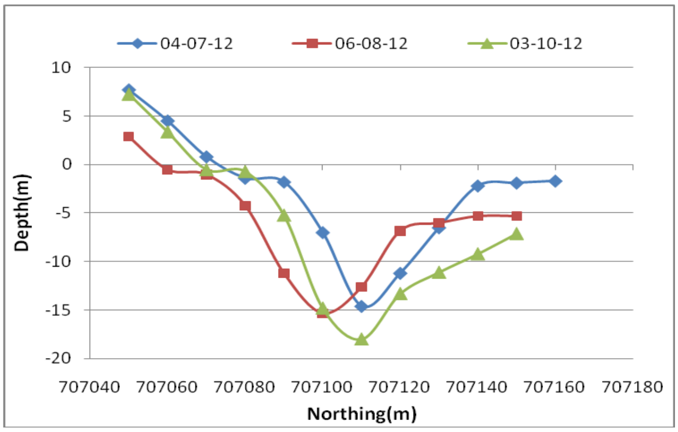

Figure 19: Monthly variation of bed level at section 4 in 2012.

Fig. 14 shows monthly variation of bed level at section 1 in 2010. Bed level changes abruptly for different months at section 1 in 2010. The maximum variation of scour depth is found 11 m (36 ft) among different months**.** Fig. 15 shows monthly variation of bed level at section 3 in 2010. Small variation of bed level is observed for different months at section 3 in 2010. The maximum variation of scour depth is found 6 m (20 ft) among different months**.** Fig. 16 shows monthly variation of bed level at section 4 in 2010. A deeper scour hole is developed by the oblique flow at 3rd October, 2010. The maximum variation of scour depth is found 6 m (20 ft) among different months. Fig. 17 shows monthly variation of bed level at section 1 in 2012. Large bed level variation is observed for different months at section 1 in 2012. The maximum variation of bed level is found 9 m (30 ft) among different months. Fig. 18 shows monthly variation of bed level at section 3 in 2012. The change of bed level of different months is gradually rises. The maximum variation bed level is found 8 m (27 ft) among different months. Fig. 19 shows monthly variation of bed level at section 4 in 2012. Bed level gradually rises due to sediment settlement. The maximum variation bed level is observed is found 5 m (17 ft) among different months.

3.2 Seasonal Variation of Bed Level

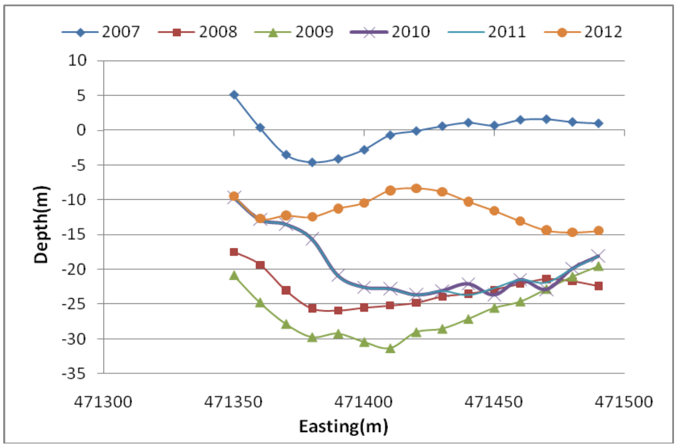

Figure 20: Variation of bed level at section 1 in monsoon.

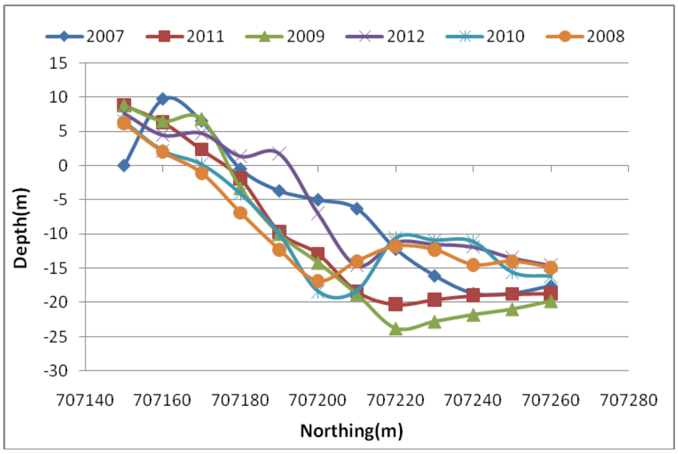

Figure 21: Variation of bed level at section 3 in monsoon.

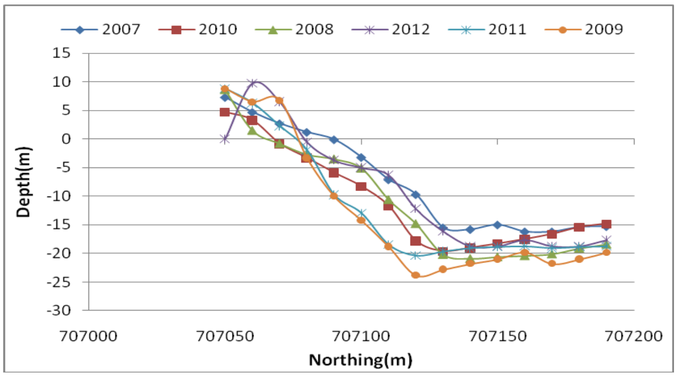

Figure 22: Variation of bed level at section 4 in monsoon.

Fig. 20 shows the variation of bed level at section 1 during monsoon for the years 2007 to 2012. Here, maximum variation of scour depth varies from 5 m to 31 m. Fig. 21 shows the variation of bed level at section 3 during monsoon for the years 2007 to 2012. Here, maximum variation of scour depth varies from 10 m to 25 m. Fig. 22 shows the variation of bed level at section 4 during monsoon for the years 2007 to 2012. Here, maximum variation of scour depth varies from 15 m to 24 m. From above graphs of monthly and seasonal variations, it is observed that, variation of bed level at section 1 is much higher compared to sections 3 and 4. Therefore, it can be concluded that, section 3 and section 4 are quite stable against scour, whether section 1 and its downstream is scour prone area. For this reason, the failure of hard point are found to be occured at the downstream region of section 1.

3.3 Maximum Scour Depth For Failure of Hardpoint

Table 1: Failure history of Sirajgonj hard point with maximum scour depth

Date of failure of hardpoint	Date of maximum scour (Section 1)	Scour depth(m)
———	01-11-2007 (4-7-2007)	10.5 (5)
———	02-01-2008 (2-7-2008)	10 (7)
10-7-2009	11-7-2009	20
17-7-2009	19-7-2009	24
19-7-2010	11-7-2010	25
18-7-2011	02-7-2011	18
21-7-2011	11-7-2011	24
———	01-11-2012 (4-7-2012)	19 (12)

Table 1 shows good relation between failure of Sirajgonj hardpoint with maximum scour depth at nearby Jamuna river. It is found that, in most of the years maximum scour depth is observed in the month of July. For this reason, failure of hardpoint occurs in the month of July. Hardpoint failure is observed in 2009, 2010 and 2011 where the maximum scour depth ranged from 18 m to 25 m. However, in 2007, 2008 and 2012 the scour depth varies from 10 to 19 m, which was not significant to cause the failure of hardpoint.

3.4 Design Considerations

The total length of Sirajgonj hardpoint is about 2550 m. The thickness of the launching apron is 1.93m. Design flow velocity is 3.7m/s. The length of the apron is 1.5 times the scour depth (below the apron setting level). The crest level is (+) 16.75mPWD, whereas approximate ground level is (+) 13.00mPWD. The side slope of the hardpoint is 1V:3.5H. Some design parameters of straight portion and upstream termination of Sirajgong hardpoint are shown in Table 2. The length of an apron on upstream termination and thickness are 19.5m and 3.83m respectively. High Flood Level (HFL) and Low Water Level (LWL) are considered (+) 15.75mPWD and (+) 6.80mPWD respectively. Apron setting level and the deepest design scour levels are (-) 4.20mPWD and (-) 13.25mPWD respectively. Figure 23 shows the cross section of Sirajgonj hardpoint at straight portion.

Table 2: Design parameter of the Sirajgong hardpoint

Section	Depth (m)	Block size (m)	
		Upper slope	Lower Slope
Straight portion	29	55 cm cubic block (one layers)	55 cm cubic block (two layers)
Upstream Termination	33	55 cm cubic block (two layers)	85 cm cubic block (two layers)

Ingis related the total mean depth, H_s=$\mathbf{h_s+h_0}$ **to the regime depth of Lacey, relying heavily on Indian data:**

$$H_s = \mathbf{0.47(Q/f)^{1/3}} \quad (1)$$

$$\mathbf{h_s} = x\mathbf{H_s\text{-}h} \quad (2)$$

where, *f* is the silt factor calculated as $\mathbf{f=1.75\sqrt{d_{mm}}}$. **To estimate the maximum scour depth, a multiplying factor (Table** 3**) must be applied, provied by Indian code.**

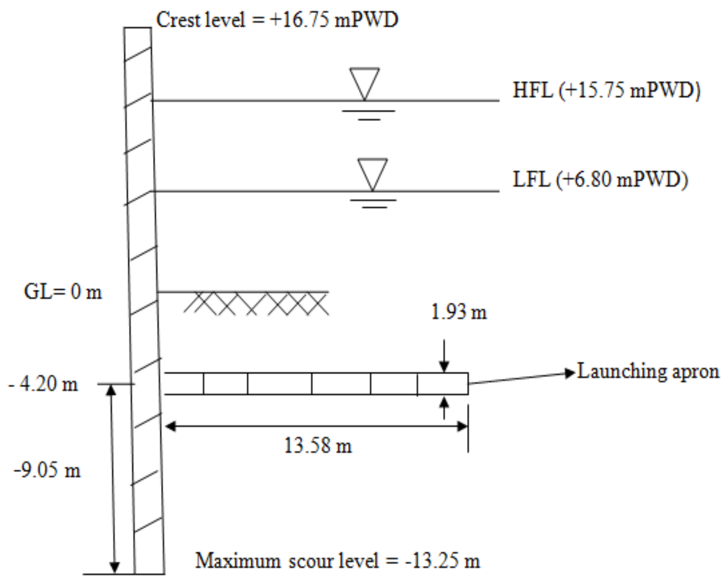

Figure 23: Cross section of Sirajgonj hardpoint at straight portion

Table 3: Multiplying factor for scour depth computation

Nature of location	Factor
Straight reach of channel	**1.25**
Moderate bend	**1.50**
Severe bend	**1.75**
Right angle turn*	**2.00**
Nose of piers	**2.00**
Along walls,abutment	**2.25**
Noses of guide banks	**2.75**

*For meandering river it may be 3.0 for massive structure.

Considering the 1190 m width channel in the Jamuna river whose bank is used to construct the structure, partical daimeter of river bed, dmm =0.20 m (Julien, 2002) and for the flow parameters mentioned above, the design of the revetment wall is checked using Ingis equation of Lacey's regime depth using Eq. (2). Calculated scour depth for the different types of revetment wall is presented in Table 4 using corresponding multiplying factor (*x*). It is shown that the factor along walls/abutment is, *x*=2.25 for which the calculated scour depth is found as 25.43 m. Since, the deepest design scour level is 13.25 m, failure of harpoint may occur if the scour depth is more than 13.25 m. It is observed that the provided depth of revetment wall (15.75 m) is much lower than requied design depth (25.3 m).

Table 4: Calculated **scour depth for different multiplying factor**

Scour depth(m)	**x=1.25 (Straight reach of channel)**	**x=2.25 (Along walls/abutment)**	**x=3 (Massive structure)**
$h_s=xH_s-h$	**7.13**	**25.43**	**39.15**
Design check	**Ok**	**Not Ok**	**Not Ok**

4. CONCLUSIONS

The study was conducted based on the secondary bathymetry survey data collected for Sirajgonj hardpoint site of Jamuna river for the years 2007 to 2012. It is observed that, variation of bed level at section 1 is much higher compared to sections 3 and 4. Section 3 and section 4 are quite stable against scour, whether section 1 and its downstream is scour prone area. For this reason, the failure of hard point are found to be occured at the downstream region of section 1. Analyzing data for the years 2007 to 2012, it is found that the yearly maximum scour depth varies from 10 m to 25 m. For most of the cases, the maximum scour depth was observed during the month of July. It is found that, the damages of hardpoint were occurred in the month of July for the maximum scour depth ranged from 18 m to 25 m. It is observed that, there is a good relation between failures of hardpoint with maximum scour depth. The design of the revetment wall is checked using Ingis equation of Lacey's regime depth, and it is observed that the provided depth of revetment wall is much lower than the requied design depth.

ACKNOWLEDGEMENTS

The authors are grateful to the Bangladesh Water Development board (BWDB) for providing necessary bathymetry survey data, design drawings and feedback to conduct the study.

REFERENCES

Rahman M. M., Arifur R. M., Munsur R. M. (2012a). Effectiveness of River Training Structures in Bangladesh, 6th International Conference on Scour and Erosion (ICSE), Paris.

Rahman M. M., Mahmud F. and Uddin M. M. (2012b). Effect of Sand Bars on Failure of Bank Protection Work along Large Sand Bed Braided River, 6th International Conference on Scour and Erosion (ICSE), Paris.

Julien, Pierre Y. (2002). River Mechanics, Cambridge University Press, Cambridge.

Uddin, M.N. and Rahman, M.M. (2009). Flow Field around Sirajganj Hardpoint along The Jamuna River in Bangladesh, 2nd International Conference on Water and Flood Management (ICWFM), Dhaka, Bangladesh.

APPLICATION OF DMA CONCEPT FOR KHULNA CITY WATER SUPPLY AND DISTRIBUTION SYSTEM

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ABSTRACT

Residents of Khulna city have been suffering from limited access to safe potable water.Only 18% of more than 1 million population in the city have access to piped water supply and the rest resorts to alternative water sources such as river, pond and shallow tube well etc.Shallow tube well water often contains significant salinity. The existing distribution network is old and poorly maintained, resulting in low quality of water in tap.The concept of DMA management was first introduced to the UK water industry in early 1980s,where a district is an area of a distribution system which is specifically defined, e.g. by the closure of valves, and in which the quantities of water entering and leaving the district are metered.The subsequent analysis of flow particularly of the night flow calculates the level of leakage within the district. This is to determine not only whether work should be undertaken to reduce leakage, but also to compare levels of leakage in the different districts to assess where it is most beneficial to undertake leak location activities(By analysing deta measured by this phenomenon leackage in the pipes can be easily detected thus necessary steps are taken as follows).The key to effective management of leakage using DMAs is to have a clear understanding of the theory of leakage. Leakage is split into two main components such as background leakage and annually occurring bursts (sometimes referred to as breaks).Background Leakage is the aggregation of sources of loss from all fittings of the network that are individually too small to be detected.The role of DMA management is to divide the distribution network into manageable areas or sectors into which the flow can be measured to determine whether bursts are present or not . The duration of water being lost is kept to a minimum by analysing the flow data so that the leakage practitioner can be aware of bursts being occurred as early as possible.What is common to the introduction of pressure management and DMA management is the requirement to define the area of the network, to close the boundaries and to measure the inflows and outflows whether for DMA analysis or to control inlet pressures. Clearly where the topography dictates, the planning of Pressure Management Areas (PMAs) and DMAs should be undertaken as one overall concept, although implementation of one stage may come before the other.As work progresses and leakage is reduced in the established DMAs, the purpose of the analysis switches to a monitoring role, where the flow into the DMA is monitored to ensure early identification of new bursts, which will trigger the need for new leak location work.When fully developed this analysis will enable a leakage practitioner to monitor a large number of DMAs effectively in Khulna city.

Keywords: *Water Supply, DMA, Distribution System Model, Khulna City, GIS,PMA*

1. INTRODUCTION

Khulna is the third largest City in Bangladesh and covers an area of 45sq kms which includes the thanas of Daulatpur, Khalispur, Sonadanga and Khulna Sadar. The built up area consists of approximately 46% residential housing, 5% industrial land and 5% commercial area. The remaining built up land includes roads and transport facilities, government and community uses, parks and open space and defence land/facilities.KCC have submitted a formal proposal to the GoB to extend the City boundaries to match the entire area included in the Khulna Master Plan area located to the West of the Rupsha River, up to the by-pass road.Accordig to 2001 census the recorded population for Khulna City was 770,498. The current population is estimated to be nearly

971598 and this is expected to grow to 1108593 within 2015 and 1238853 by 2020[1]. This assumes a conservative annual growth rate of 2.35% per year. In the event that the regional growth potential of the area can be achieved (Chapter 2 refers), then it is plausible that the growth rate would be significantly higher. In 2005, the population in informal housing was estimated at just under 189,000 (37,800 households). This equates to approximately 20% of total Khulna population. Informal housing densities are very high at around 132,000 persons per sq km. Thousands of households get affected at different hotspots of Khulna city.So proper planning is needed to supply them sufficient water with DMA concept.

2. DMA CONCEPT

DMA is a small discrete area with its own water supply system and distribution network for a community and isolated from remaining network without affecting supply system of other areas.supply source for a DMA may be groundwater (ptw) and/or surface water.water balance in a DMA can be accounted from its source capacity and consumption.

2.1 Benefits of DMA Concept

- ✓ up-to-date water balance information
- ✓ Minimized non-revenue water (nrw).
- ✓ Easy detection of leakages and illegal connections.
- ✓ Energy efficient system.
- ✓ pressurized system
- ✓ Improved water quality.
- ✓ Better customer satisfaction.
- ✓ Turning KWASA into a profitable organization.

2.2 Criteria for DMA Concept

Primary Criteria:

- Hydraulic Isolation
- DMA Size : Neither Too Big Nor Too Small
- At Least One Source/PTW
- At Least One External Connection for Emergencies
- All Connections and Sources must be Metered.

Secondary Criteria:

- Consideration of Well Defined Roads
- Administrative Boundaries
- Land Use and Housing Pattern
- Future Developments

2.3 DMA Features

Most of the distribution pipes within a DMA are to be rehabilitated with new HDPE pipe and the distribution network of all DMAs will be designed for future. Also the network is designed for surface water scenario also, all the zones are need to be metered and there will be no illegal connection. In distribution pipe minimum pressure to be ensured within DMA at all-time is above 10m. Satisfying minimum pressure, surplus water of a DMA (if any) will be delivered to the transmission line or to nearby DMAs through inter-DMA pipes where exists negative water balance.The supply source will eventually shift from groundwater to surface-water.

2.4 DMA operations:

Major Control Parameters are

- Pump Operation
- PSV and/or PRV Operation at Bulk Meter Chamber
- Bulk Meter and Valve Efficiency
- Domestic Meter and HC Characteristics

[1] Population estimates based on BBS,2008 Statistical Pocket book of Bangladesh and key indicators sample vital registration system (SVRS), www.bbs.gov.bd

3. METHEDOLOGY

3.1 Study Area

Residents of Khulna city are suffering from serious water crisis as Khulna Water and Sewerage Authority (Wasa) is currently supplying only nine crore litres of water against the daily demand of 24 crore litres for around 15 lakh people of Khulna city.

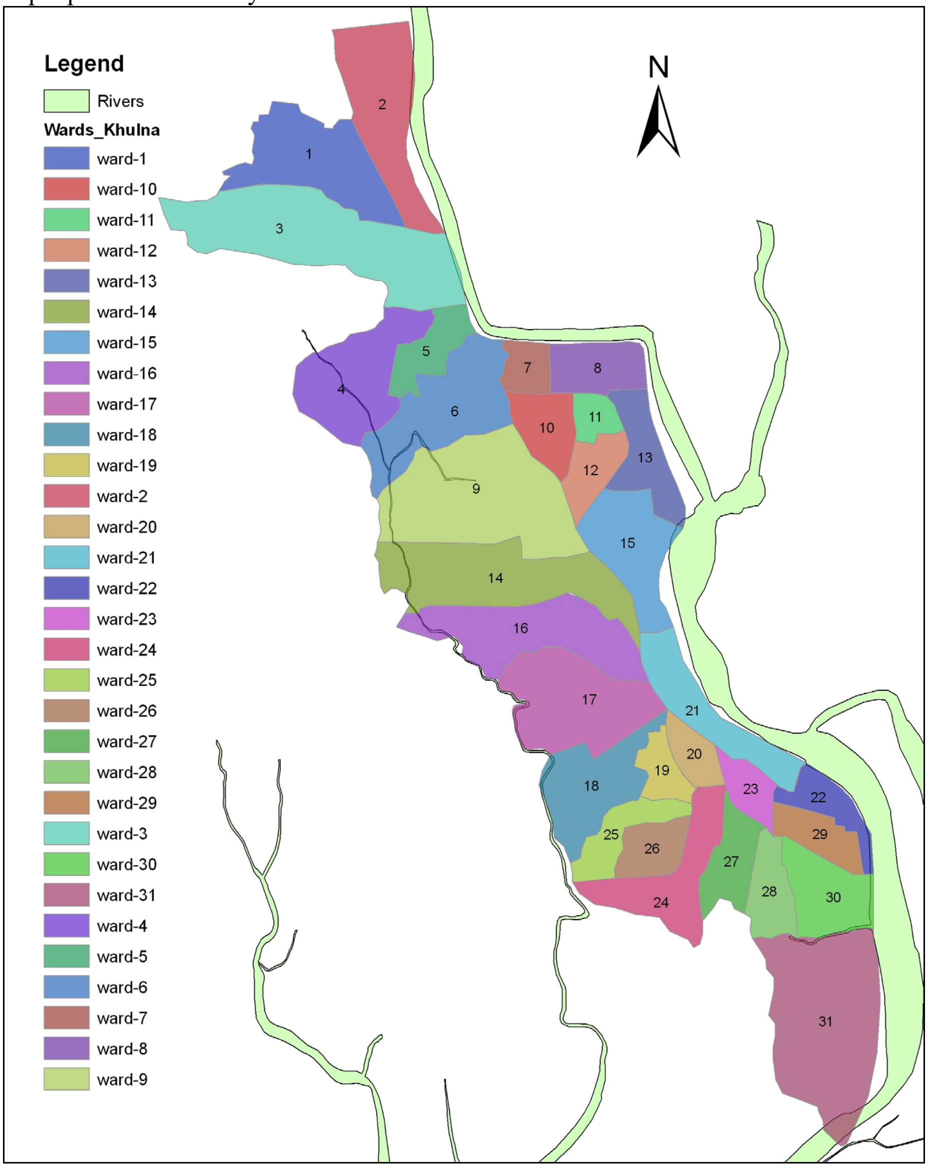

Figure 3.6: Khulna City Corporation wards boundary.

3.2 Climatic Condition

Khulna city experiences Indian Ocean monsoon climatic conditions. The area experiences four meteorological seasons; pre-monsoon (March to may), monsoon (June to September), post-monsoon (October to November) and dry (December to February). Average annual rainfall is in the range of 1,700 to 2,200 mm. about 70% rainfall occurs during the period from June to September. Mean monthly rainfall during the same period is between 300 to 450 mm. maximum daily rainfall is about 200 mm but it is increasing. In 2004, the maximum daily rainfall was 364 mm. average temperature range is between 25ºC to 31ºC. Maximum temperature may rise up to 40ºC and may go down to 6ºC. Average humidity remains at 80% to 90%.

3.3 Topography

The city area, the central part of the greater Khulna is occupied by the southern half of the Sundarban tract. The rest of the area is covered by the floodplain of the jamuna, the Padma and the Meghna Rivers. A remarkable difference in the ground elevation can be observed throughout the Khulna city, which is reflected in distinct landforms: high lands, low lands, and abandoned channels and depressions. Ground elevation of the city varies from 0.5 m to 12 m (pwd). About 60% and 70% of the city area includes low lands, abandoned channels and depressions and the elevation of these areas vary from 0.5 to 5 m (pwd).

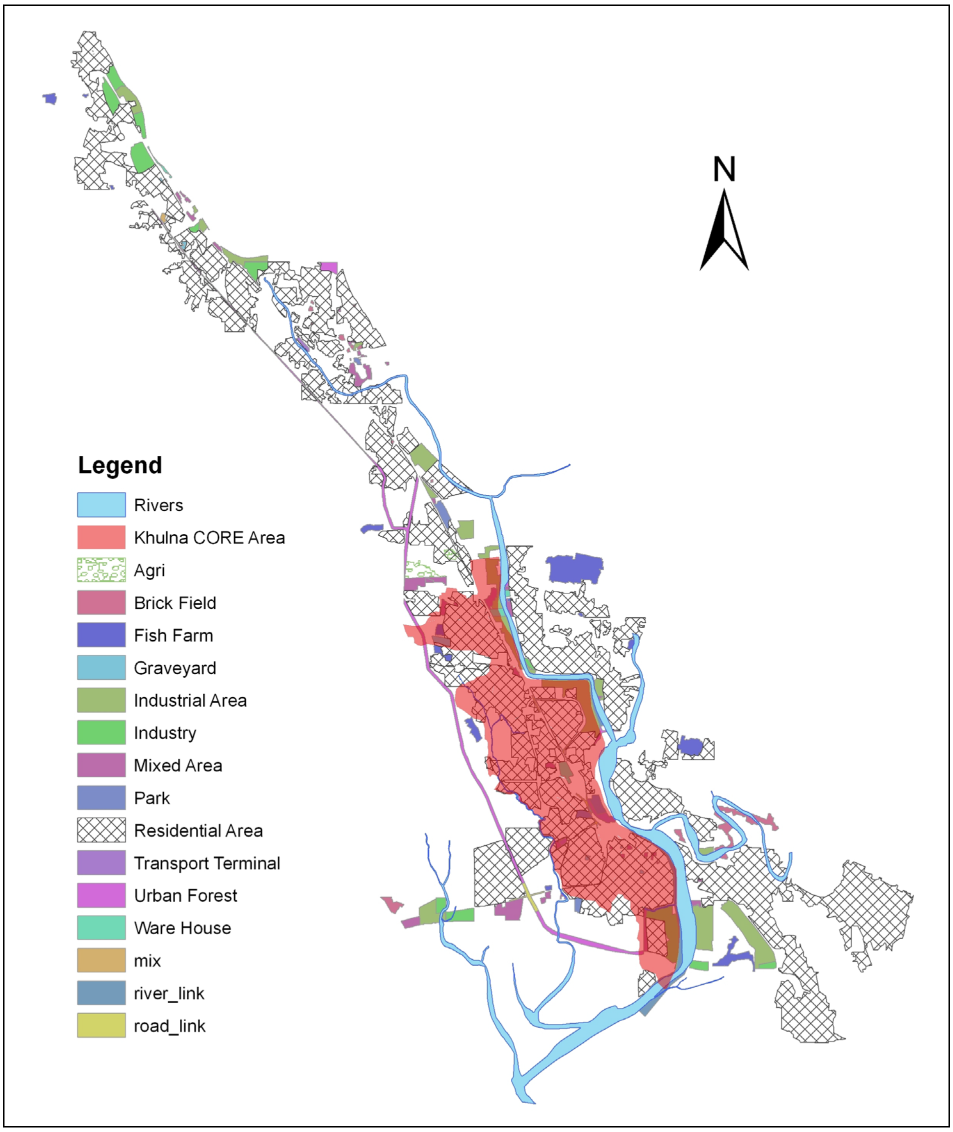

Figure 3.2: Land use map of Khulna City core area

Thousands of households get affected at different hotspots of Khulna city. Hotspots which get seriously affected by water supply problems include Diyana School area under ward no. 4, Thana Doulatpur; Khalishpur, ward no. 08, Thana Khalishpur; water congestion affects Khulna Medical Collage, Ward: 17, Thana Sonadanga; Sonadunga Bus Stand, Ward no 18, Thana Sonadanga; Duckbanglo, Ward: 21, Thana Sadar; Feerighat, Rupsha, Ward no 21, Thana Sadar; flood affects Sangita More, Ward: 25, Thana Sadar; and Hotel Royel More, Ward: 25, Thana Sadar; drainage problem affects Shib Bari More, Ward 25, Thana Sadar; and submerged with water with heavy rainfall or drainage problem affects Forizi Para, Ward: 28, Thana Sadar. The inhabitants suffer in rainy season and it becomes more severe when water congestion remains for 1-2 days to about 1 to 2 month. Almost all people suffer in rainy season.

People in general do not take much initiative to overcome the problems as they consider it beyond their capacity but sometimes they try to overcome it collectively. Usually they don't take any preparation as because the whole area gets affected and they do not have capacity to handle it. They go to the leaders but do not get support to overcome the problem. City dwellers face serious problem during the crisis like diarrhoea, cholera, dysentery, typhoid, etc. During the problem they do not move to other places as there is no alternate water supply options..

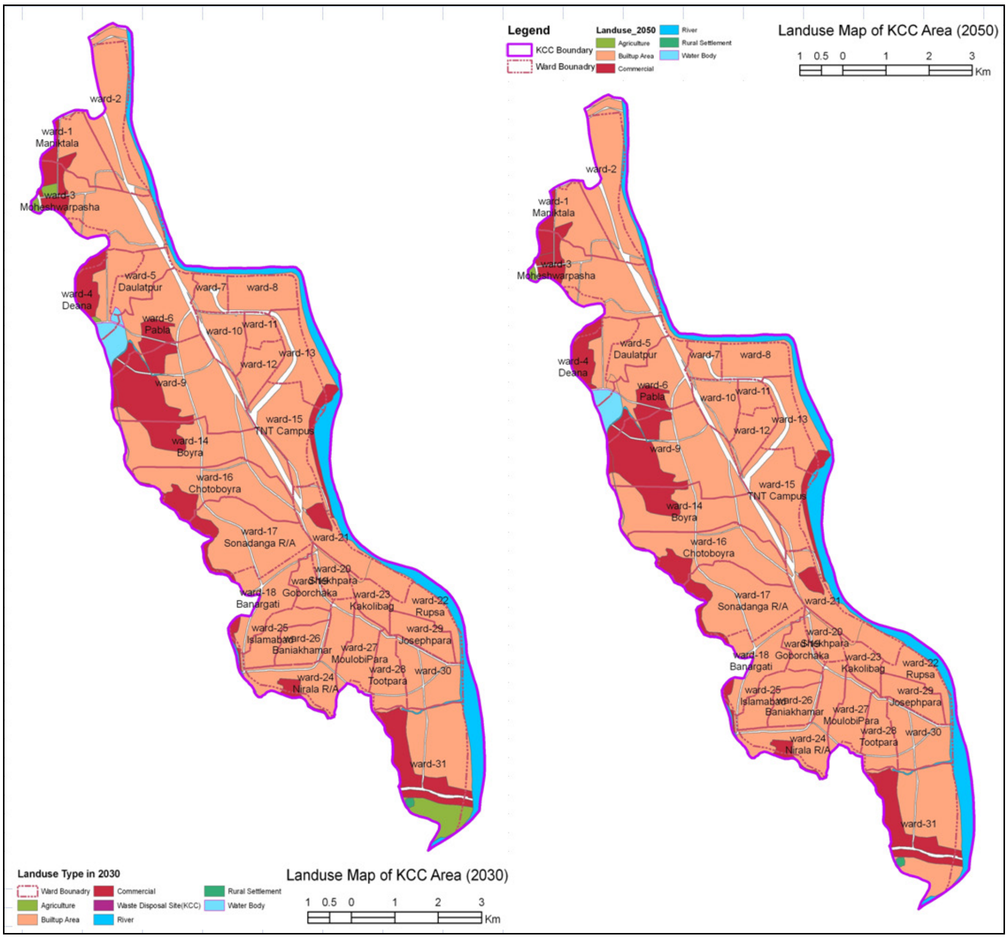

Figure 3.3: Future change of Land use map of Khulna City core area

3.4 Meterological Status of Khulna

The Khulna climatic conditions are summarized in the Table 3.1

Average 1995-2004	Mean temp. (ºC)	Max. temp. (ºC)	Min. temp. (ºC)	Mean humidity (%)	Precipitation amount (mm)	Mean wind speed (m/s)
Jan	18	26	13	69	11	0.3
Feb	21	28	15	63	23	0.3
Mar	26	32	20	56	64	0.5
Apr	29	33	23	70	126	0.6
May	29	33	24	80	335	0.3
Jun	28	32	26	85	290	0.3
Jul	28	31	26	85	377	0.3
Aug	29	31	26	85	325	0.3
Sep	29	32	26	84	293	0.3
Oct	27	31	24	79	167	0.2
Nov	24	29	20	70	31	0.3
Dec	19	26	14	74	12	0.2
Total					2063	
Average	25.6	30.3	21.4	75		0.33

Seasons of Khulna city can broadly be classified into 3 categories namely winter, summer and rainy season. Rainy season mainly spans from May – September with an average rainfall 137 mm (May) to 248 mm (September), highest in August 337 mm.

3.5 Water sources

About 15 crore litre of water is being supplied through 3748 tube-wells installed by Khulna City Corporation (KCC) and 22,701 other private owned tube-wells.And so, a good number of city dwellers have to depend on other sources including ponds for water. Khulna Wasa has 73 large 'production' tube-wells which are used to supply nine crore litres of water through pipelines for the subscribers in Khulna city, officials said. Khulna Wasa has undertaken a Tk 2,500 crore 'Khulna Water Supply Study' that has already got approval from the Executive Committee of National Economic Council for implementation.The water brought from the Madhumati will be purified at the water treatment plant at Shamantashena and supplied through pipelines to the subscribers of Khulna city.The proposed water supply study includes installation of 700 kilometres of pipeline.Works for acquiring land under this study are going on for construction of a water treatment plant.At least 75,000 subscribers will get benefit of water supply with implementation of this study which is expected to be completed by 2017. After completion, it will be possible to supply more five crore litres of water a day in Khulna city.

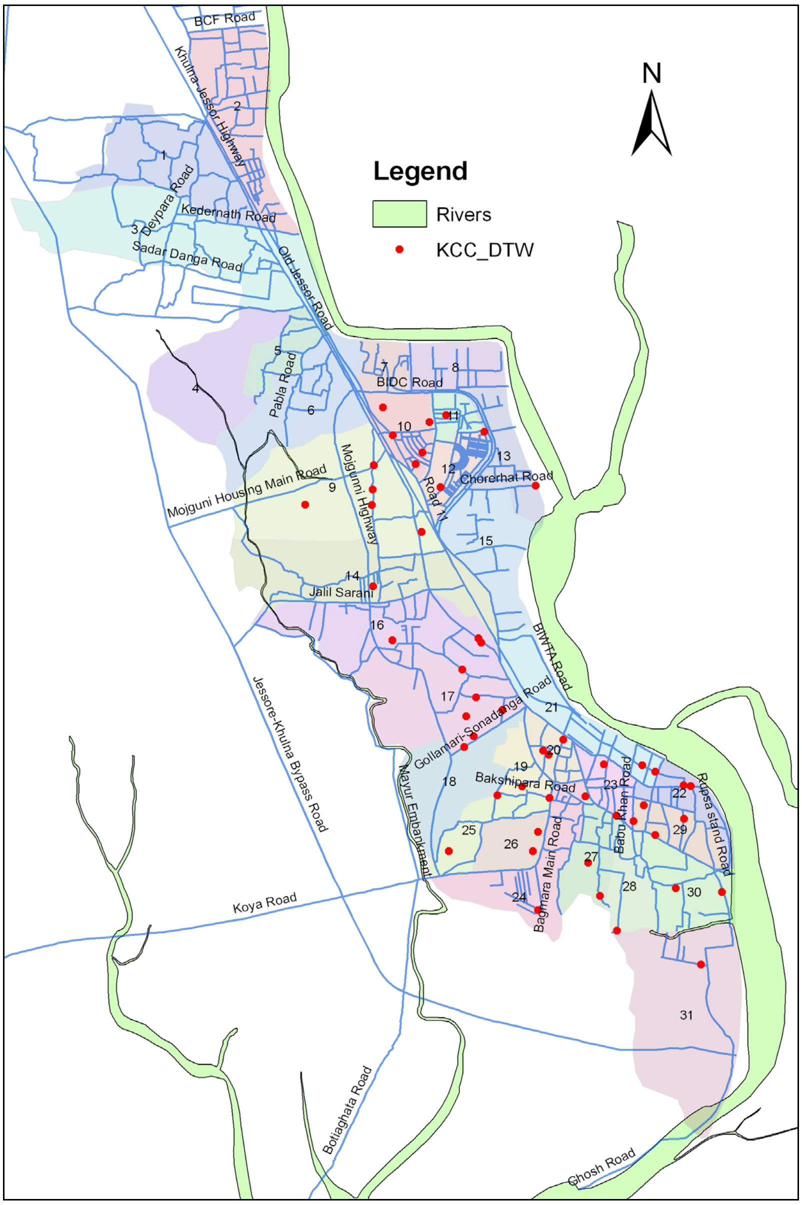

Figure 3.4: Location of Khulna City Corporation Deep Tubewells

3.6 Existing Water Supply System

3.6.1 Water Sources

The network in Khulna is currently supplied by limited surface water and ground water from tubewells across the city. The surface water is treated and chlorinated whilst the ground water is generally not. The tube wells are often poorly constructed and have a relatively short life. They are rarely rehabilitated but rather, new tube wells are drilled, often adjacent to the old. The ground water table is being substantially lowered by abstraction in excess of the sustainable yield. Surface water use is limited due to the lack of suitable, non-polluted water.

Source of drinking water: Data (Table 3.2) on source of drinking water show that Khulna have highest number of tap water use while again deep tubewell use also very high in Khulna. In case of other towns dependency on other source is much higher than Khulna. Interestingly there is no dependency on open water use which means some kind of arranged water use is practiced. In 2030 and 2050 dependency on tap water will increase further which means more use of tap water will be there. Shallow tubewell use in Bagerhat is less as there is more salinity in upper level of groundwater.

Source of water for domestic use: Again source of domestic water use (Table 3.2) is different in Khulna from Bagerhat and Sathkhira. Khulna has more dependency mainly on deep tubewell and tap water while other two towns have different scenario. Reasons for having different scenario in those towns are mainly because of low DTW than Khulna. In 2030 and 2050 there will be more dependency on tap water like Dhaka.

Table 3.2: Water sources in Khulna City

Source of drinking water	Khulna District	Domestic Water Source	Khulna District
Tap water	10.00%	Tap water inside house	15.80%
Tap water in front of the house	0.30%	Tap water in front of the house	1.30%
Govt. tap pipe/ road side tap	3.00%	Govt. tap or tap at road side	4.70%
Shallow tubewell (<500 ft)	13.70%	Shallow tubewell (< 500 ft)	12.20%
Deep tubewell (500+ ft)	72.80%	Deep tubewell (<500+ ft)	63.30%
Pond/ irrigation canal	0.20%	Dug well	0.00%
Bottled water	0.01%	Surface water (pond, canal etc.)	2.70%
Total	100.00%	Total	100.00%

3.6.2 Network

There are over 227 kilometres of existing pipe lines comprising of AC, MS, and PVC. The old large dia pipes are envisaged to be in poor state and needs to be replaced. However, some of the uPVC pipelines installed in the last decade may be in a good form to be used further. This can only be ascertained after a detailed investigation and assessment. uPVC pipes in the diameter range 200 mm – 100 mm may be considered for reuse, if found in a good and usable state. But, smaller diameter service pipelines are likely to be disturbed and damaged during the implementation process of the ensuing loan and, hence, will have to be largely replaced.

Table 3.3: Details of Existing Pipelines in KWASA

Sl No.	Diameter (mm)	Approximate Length (km)	Material
1	75	2.62	PVC
2	100	110	PVC
3	150	66	PVC
4	200	22	(MS, AC 6 km)
5	250	10.55	MS, AC
6	300	16.30	MS, AC
Total		**227.47**	

Source: JICA FS Report, Khulna Water Supply Improvement Project

The water supply system in Khulna has grown organically over the years. Heavy reliance on ground water and a lack of coherent resource planning has resulted in a fragmented system which is under sized in some areas and over sized in others. There is no clear distinction between transmission mains and distribution mains which means laterals and reticulation are often connected to large diameter pipes resulting in loss of pressure and increased leakage.The system is pressurised by the submersible pumps in the tubewells with little or no use of overhead tanks for system balancing and pressure conservation. There is no pressure zoning and the lack of design of the tubewell pumps results in very in efficient operation of the network.The network is built from a range of Asbestos Cement (AC), Ductile Iron (DI), Steel (MS) and PVC pipe. The majority of newer pipe is PVC. Jointing is generally by cement joints. Leakage in the network is estimated to be in excess of 40% due to leaking joints (PVC pipes) poor quality house connections and poor construction of the original system. It is likely that the PVC suffers from rapid crack propagation as a result of frequent pressure cycling following pump failure and water hammer conditions.

3.6.3 Zones and DMA's

The networks need to be divided into some zones for operational purposes. Under this study, the zones will need to be divided into district metered areas (DMA's). These DMA's will be isolatable from the rest of the network and will allow both pressure and flow management.

3.6.4 Data Capture & Metering

There is no data capture (bulk flow meters or pressure gauges) within the present network. Bulk meters exist at each production tube well although these are frequently out of service.Need some other data for review like:

- Water Supply System and Sewerage System Drawings
- Drainage System Drawings
- Topographic Maps
- Network Analysis and System Metering Including Zone Demarcation
- Deep Tube Well (DTW) Data

3.6.5 Chlorination & Water Quality

The surface water plants are equipped with gas chlorination equipment. Tube wells are in some cases equipped with basic chlorination equipment although this is generally unsafe and often no chlorine gas is available. The level of chlorination within the Khulna network is insufficient for disinfection and, given the likelihood of water ingress due to low pressure the water quality often falls below standards required for potable water.

4. PROPOSED WATER SUPPLY SYSTEM

4.1 4.1 Study on of water demand

Demand for Water for KCC area has been estimated by the JICA study using the following methods:

- Population studyion is made using a 2% growth rate in Khulna urban population
- Changes in the size of household by 0.04% per year using census data (2001-2011)
- Per capita water demand is fixed at 150 LCD
- 20% of the water is unaccounted.
- 25% of the water is for non-domestic use.
- Population distribution by ward remains unchanged assuming that the increased population is evenly distributed across the various wards.

DMAs are often used as a tool to control and drive down leakage in networks that have received little or no leak location work other than dealing with reported occurrences. Initially the DMAs will be used as a tool to determine which parts of the network are experiencing the highest level of leakage and to discount areas where there is limited leakage, so that resources can be targeted to the greatest effect.As work progresses and bursts are located and repaired, the success of this work can be measured at a local level, as initially the impact of the work is unlikely to be perceived within the larger network. Clearly where the topography dictates, the planning of Pressure Management Areas (PMAs) and DMAs should be undertaken as one overall concept, although implementation of one stage may come before the other. As work progresses and leakage will be reduced in the established DMAs, the purpose of the analysis switches to a monitoring role, where the flow into the DMA is monitored to ensure early identification of new bursts, which will trigger the need for new leak location work. In many instances where the ongoing requirement of DMA management has not been implemented, the early leakage savings have been lost due to the occurrence of these new ongoing bursts not being dealt with.

Table 4.1: Study on present and future Water Demand under business as usual scenario

	2009	**2010**	**2015**	**2020**	**2025**	**2030**	**2050**
Population	957,000	976,000	1,078,000	1,190,000	1,314,000	1,450,000	2,155,000
lpcd	109	110	120	140	140	150	150
Water for non-domestic use	25%	25%	25%	25%	25%	25%	25%
Unaccounted for water	30%	30%	20%	20%	20%	20%	20%
Water Demand							
Domestic	104	107	129	167	184	218	323
Non-domestic	26	27	32	42	46	55	81
Unaccounted for water	39	40	32	42	46	55	81
Total	169	174	193	251	276	328	485

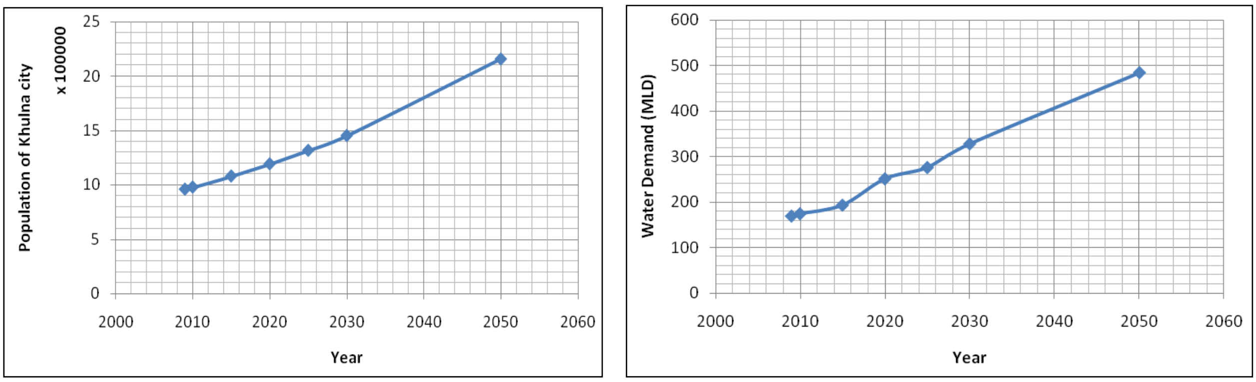

Figure 4.1: Water Demand for Khulna City (2010-2030 – JICA, 2050 IWM)

Table 4.2: Study on of Water Demand

		Baseline Trends			
	Year	Population	Households	Water Demand (MLD)	LPCD
JICA	2009	957,000	217,500	169	109
	2010	976,000	223,900	174	110
	2015	1,078,000	259,100	193	120
	2020	1,190,000	300,500	251	140
	2025	1,314,000	349,500	276	140
	2030	1,450,000	407,300	328	150
Studyn I	2050	2,155,000	1,346,900	485	150
Studyn II	2,050	2,155,000	1,346,900	517	160

4.2 Options for Future Water Supply

One area that is not always fully linked to DMA management is that of pressure management. Julian Thornton's article Managing Leakage by Managing Pressure (*Water21*, October 2003) clearly identified the relationship between leakage and pressure and the potential leakage savings to be made.What is common to the introduction of pressure management and DMA management is the requirement to define the area of the network, to close the boundaries and to measure the inflows and outflows- whether for DMA analysis or to control inlet pressures.

Sea water level rise may cause hazard: Sea water level rise may causes hazard is being perceived by most people. Some 91.5 per cent opine that it would cause hazard to them and 90.1 per cent anticipate risk. Out of that highest per cent (91.1%) of risk of homestead going under water was anticipated in around khulna city, reduce income (18.8%) in Khulna, and water logging, crop damage, damage of life and assets, etc. were also perceived. In 2030 and 2050 those are likely to increase further.

Table 4.3: Add-on options for future water supply system in Khulna core area

Add-on option Description	Needed Intervention	Social concerns and present adaptation mode	Recommendations
Apply DMA concept to reduce leakage	-No supply line damage by people -Immediate information to KWASA	-At present people are less concerned about that as most of them are dependent of DTW - More awareness will require as DWASA has started campaign	Create more awareness and proper implementation works
Water Demand Management	-Progressive water pricing -Awareness raising campaign including promoting water saving equipment /low water use toilets, washing machines, etc. -Managing public sentiment /resistance related to increased water pricing -industrial water use reduction	-People are willing to pay more if they get regular water supply -Awareness depends on availability of water (The higher is the water supply, the lower is the awareness) -At present any serious public sentiment is not there centering water supply -Industrial water use almost cent per cent managed by the enterprise itself.	Progressive water pricing may be introduced. Awareness raising
Dual Reticulation (2 pipe system)	-Progressive water pricing -Include awareness raising campaigns -Need to educate people on how to use the dual system	-Willing to pay if supply is ensured -People are not aware of it -People are using partially a kind of dual system now as they use DTW water for drinking purpose and supply water for domestic purpose.	Awareness raising

5. RECOMMENDATIONS

Best practice analysis of DMA flows, requires the estimation of leakage when the flow into the DMA is at its minimum. This typically occurs at night when customer demand is at its minimum and therefore the leakage component is at its largest percentage of the flow.Techniques are now available to analyse the minimum night flow to estimate the level of leakage and additionally to split this estimate into background. The analysis of leakage is based on the minimum night flow, which can be recorded and analysed continuously night after night with the use of data loggers and appropriate software. Based on this study, relevant reports of estimated leakage from bursts in individual DMAs can be developed to provide the leakage practitioner with a schedule of leakage that can be reduced. This reduction can be represented as a volume of water, a potential number of bursts that can be found, an estimate of the cost of leakage that is being lost, or a ranking system developed to suit local conditions.

6. CONCLUSIONS

When fully developed this analysis will enable a leakage practitioner to monitor a large number of DMAs in Khulna City core area effectively and focus work in key DMAs, which will generate most benefit from leak location.The level of leakage can be further confirmed by a 'top down' assessment of leakage. This analysis requires an assessment of customer use, which is subtracted from the total flow into the area to estimate leakage. In most instances this leakage volume, measured over a period of 6 to 12 months, will be compared with the aggregate of leakage from DMAs in the same area.Whilst the analysis of night flow represents best practice guidance, the study will outline possible interim approaches, so that analysis can be carried out in the interim to enable leak location targeting to progress whilst additional data is gathered.This may take several years to develop a well defined DMA boundary in Khulna City for better water supply and management.

7. ACKNOWLEDGEMENTS (OPTIONAL)

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REFERENCES

Modeling Report (2013) of Dhaka water Supply Sector Development Program (DWSSDP) Rehabilitation of Distribution Networks project under DWASA by DMC, IWM, AQUA and BETs.

Managing leakage by District Metered Areas: a practical approach by John Morrison (2004)

Charalambous B., Lambert A and Lalonde A (2005). Using practical predictions of Economic Intervention Frequency to calculate Short-run Economic Leakage Level, with or without Pressure Management IWA Leakage2005 Conference, Halifax (Canada)

Fantozzi, M and Lambert A (2007): Including the effects of pressure management in calculations of Short-Run Economic Leakage Levels. IWA Conference 'Water Loss 2007', Bucharest, Sept. 2007.

Fantozzi, M and Lambert A (2008): Recent developments in predicting the benefits and payback periods of introducing different pressure management options into a Zone or small distribution system IWA Conference 'Water Loss Management 2008', Skopje (FYROM), June 2008.

Kovacs, J. (2006). Case studies in applying the IWA WLTF approach in the Western Balkan Region: Results Obtained. Skopje Conference 2006: Water loss management, telemetry and SCADA systems in water distribution systems. imgd@zg.htnet.hr

Lambert A, 2002. International Report on Water Losses Management and Techniques: Report to IWA Berlin Congress, October 2001.Water Science and Technology:Water Supply Vol 2 No 4, August 2002

Lambert A and Lalonde A (2005). Using practical predictions of Economic Intervention Frequency to calculate Short-run Economic Leakage Level, with or without Pressure Management. Proceedings of IWA Specialised Conference ‘Leakage 2005’,

Thornton, J (2003) Managing Leakage by Managing Pressure. Water 21, October 2003

Thornton, J and Lambert A (2006): Managing pressures to reduce new break frequencies, and improve Infrastructure Management. Water 21, December 2006.

Thornton, J and Lambert A (2007): Pressure Management extends infrastructure life and reduces Unnecessary energy costs. IWA Conference 'Water Loss 2007', Bucharest, September 2007 (in press).

Waldron et Al. (2002) Establishing Pressure Management Zones and District Metering Areas: The Toolkit. Manual No 4 in the series 'Managing and Reducing Losses from Water Distribution.

Proceedings of the 2nd International Conference on Civil Engineering for Sustainable Development (ICCESD-2014), 14~16 February 2014, KUET, Khulna, Bangladesh, ISBN: 978-984-33-6373-2 (CD-ROM)

POSSIBILITIES OF SUCCESSFUL COASTAL FLEXIBLE REVETMENT TECHNOLOGY AGAINST RIVERBANK FAILURE

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ABSTRACT

Coastal based flexible revetments for instance revetment of geobags placed successful examples around the world. Geobags are a common geotextile product used for the construction of low cost coastal protection works. In this paper the term 'geobag' is reserved for sand filled geotextile bag, whereas the term sandbag is used to describe bags manufactured from any materials including geotextile, nylon, polyester, jute. An intensive review on the laboratory, field and numerical model studies previously was undertaken on sandbag performance in coastal protection. This presents a summary of the contribution to the body of knowledge from the available publications on the (i) bag design specification, (ii) geobag revetment construction specification, (iii) the mechanical properties of bag, and (iv) the active hydraulic forces on the structure. Sandbag of nonwoven needle – punched geotextile fabric with seam strength of at least 80 – 90% of the tensile strength of the fabric and approximately 80% fill ratio showed desirable performances in coast protection. The 50% layer–to–layer over lapping of sandbag achieved optimum revetment stability whereas the average friction angle between sandbag found to be 30°. The findings also suggested that sandbag becomes unstable from a revetment above a flow velocity of 1.5 m/s. So far the technical information available in the literature is based on the sandbag performance in coastal protection works i.e. mostly wave action on geobag structure. For riverbank the nature of revetment instability supposed to be different, however, based on the available experiences on coastal revetment this study attempted to identify the knowledge gaps need to resolve for applying geobags as a flexible revetment for riverbank protection.

Keywords: *Flexible revetment; Geobag; Sandbag; Coastal protection; Riverbank failure*

INTRODUCTION

In this paper the term geobag is reserved for sand filled geotextile bag, whereas the term sandbag is used to describe bags manufactured from any materials including geotextile, nylon, polyester, jute. Geobags are a common geotextile product used for the construction of low cost coastal protection. In addition to their use as groynes, geobags have also been used to prevent erosion and scour in bulkheads and revetments in coastal, island and bridge abutment applications (Gutman 1979; Gadd 1988; Korkut, Martinez et al. 2007). Several attempts have been made to understand the performance of sandbags (Venis 1968; Porraz, Maza et al. 1979; Kobayashi and Jacobs 1985)and geobags (Bezuijen, Groot et al. 2004; Recio and Oumeraci 2009 a; Recio and Oumeraci 2009 b) in coastal applications. Findings from physical modelling (Venis 1968; Porraz, Maza et al. 1979; Kobayashi and Jacobs 1985; Gadd 1988; Krahn, Blatz et al. 2004; Grüne, Sparboom et al. 2006; Mori, Aminti et al. 2008 ; Recio and Oumeraci 2009 a; Recio and Oumeraci 2009 b; Dassanayake and Oumeraci 2010), field studies (Heerten, Jackson et al. 2000; Pilarczyk 2000; Bezuijen, Groot et al. 2004; Saathoff, Oumeraci et al. 2007; Mori, Aminti et al. 2008 ; Corbet 2005), analytical approaches (Breteler, Pilarczyk et al. 1998) and numerical modelling (Recio and Oumeraci 2009a; Recio and Oumeraci 2009b) are tabulated in coastal geobag section.

The aim of this paper is to acquire knowledge from coastal geobag structure to develop knowledge on riverbank protection geobag structures.

COASTAL GEOBAGS

This section presents a summary of the contribution to the body of knowledge from the researches on the (i) bag design specification, (ii) geobag structure construction specification, (iii) the mechanical properties of bag, and (iv) the active hydraulic forces on the structure.

➢ Bag design specification

The bag design specification includes the physical properties of the bag materials i.e. sand and fabric. Saathoff et al. (2007) mentioned the sand d_{50} was in a range between 15 to 25 mm for the protection work in field. From the field test, Bezuijen et al. (2004) concluded that the use of dry and clean sand as a fill material can reduce the risk of tearing the bag fabric geotextile through absorbing a significant part of the fall energy while dropping the bag as practiced during revetment construction. The thickness of the fabric is also an issue, Restall et al.(2002) noted down the thickness of the geotextile employed for bag preparation in this case 5.3 mm (Stockton beach revetment) to 5.5 mm (North Kirra groyne). Table 1 summarizes the bag size used in different protection works in the field. The length of these bags was in a range of 1.22 to 2 times of its width (Table 1). The empty bag size is expressed normally by the length and breadth, as the thickness might vary with the fill ratio. The sand filling ratio got priority due to its influence on geobag performance towards the whole structure performance. Studies in the laboratory and the field suggest that the most acceptable filling ratio is 80% of the actual volume of a geobag (Figure 1). Presently more research is carrying out by Dassanayake and Oumeraci (2010) to summarize the influence of fill ratio on geobag structure performance.

Table 1: Published bag size in field practice

Geobag structure	**Bag size**			**Year**	**References**
	Length (m)	Width w (m)	Thickness (m)		
Stockton beach revetment	1.5 (1.36w)	1.1	0.4 (0.36 w)	2002	Restall *et al.*
Stockton beach revetment	1.5 (1.25w)	1.2	0.45 (0.38 w)	2007	Saathoff *et al.*
Maroochy groynes	2.2 (1.22w)	1.8	0.7 (0.39 w)		
Jumaira beach revetment	2.2 (1.22 b)	1.8	0.7 (0.39 w)		
Eider storm surge barrier	2.7 (2 w)	1.35	–		
Marina di Ronchi (submerged groin)	2.5 (1.67w)	1.5	0.5 (0.33 w)	2008	Mori *et al.*

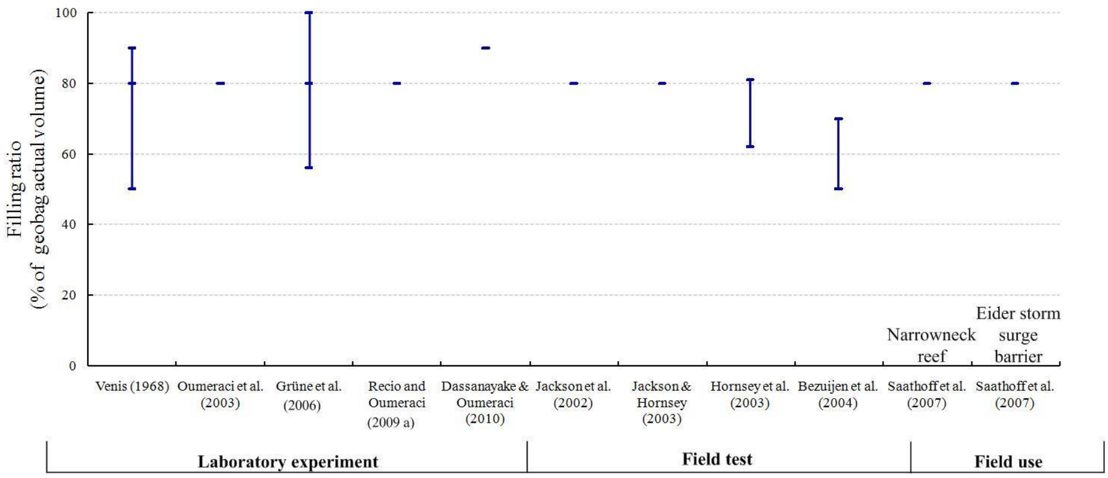

Figure 1: Published filling ratio under laboratory experiments and field experience

➢ Geobag structure construction specification

Construction specification for geobag structures mostly addresses protection work thickness, bag placement (with respect to coast/bank line) and the slope of the structure. In the Stockton beach revetment, the double layer bag thickness and the running bond bag setup (the geobags are laid longitudinally, in a typical brick wall pattern, with the geobag joints laying in the middle of the bags in the layers directly above and below) was innovative as there was no guidelines for bag placement available at this time (Saathoff, Oumeraci et al. 2007). The most effective bag placement relative to the coast/bank line is found to be parallel to the flow direction (Findings section, Table 4). Apart from the geotechnical features in the field, the geobag structure slope is controlled by the extent of bag overlapping employed and the optimum overlapping has been found to be 50%

(Table 4). From the available literatures the most commonly used slope is between 1V: 1H to 1V: 2H although slopes as flat as 1V: 10H have also been employed (Figure 2). In field the selection of slope is normally based on the design wave height and the pattern of erosion or scour hole (Heibaum 1999), so it will be variable based on location. To achieve the design bag layer, bond, placement and finally the slope, there is often a need for mechanical device. Manually bag drop introduces quality control issues and there is therefore uncertainty involved in the final position achieved. To overcome this issue, there is a well-known computer software the GeoCoPS (2.0) which is used for predicting the theoretical shape of geobag on the seabed after placement from a mechanical device(Hornsey, Jackson et al. 2003).

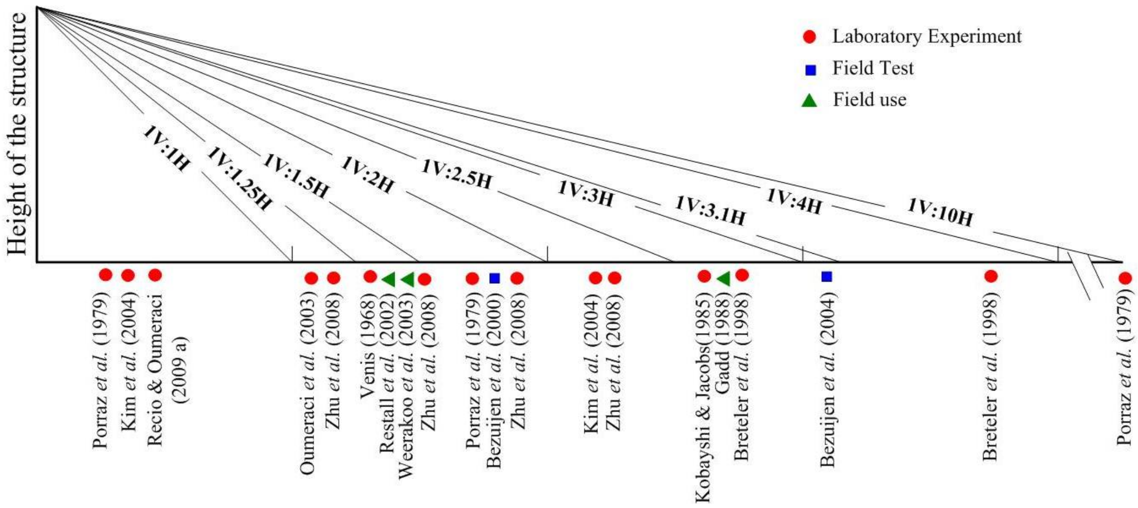

Figure 2: Published sandbag structure slope

The life expectancy of geobag structure in coastal protection work is normally 20 to 25 years (Kobayashi and Jacobs 1985; Heerten, Jackson et al. 2000; Saathoff, Oumeraci et al. 2007).

➢ Mechanical properties of bag

Mechanical properties of bags, that are the core properties to ensure the stability of a geobag construction, are the internal friction, permeability and deformation of the bag. Recio and Oumeraci (2009 a) intensively studied the permeability and deformation of bags and the effect of these properties on the structure (Table 4). Among the available literature, Kim et al. (2004) and Krahn et al. (2004) only carried out experiments using a large shear box to obtain estimates of friction angle under different loads. The average friction angle between geobags is found to be 30° (Figure 3).

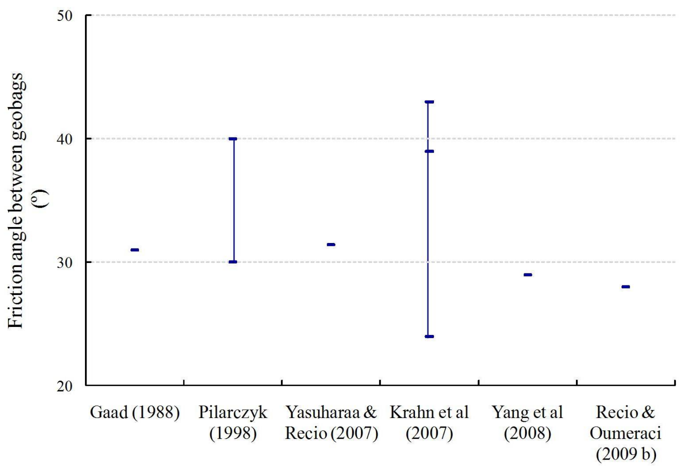

Figure 3: Published geobag–geobag friction angle

➢ The active hydraulic forces on the structure

The hydraulic loading causing instability in a structure are only noted by Pilarczyk (2000) for incipient velocity (Table 2) and Recio and Oumeraci (2009 a) for hydrodynamic forces (drag and lift) (Table 3). The term 'incipient velocity' was explained by Pilarczyk (2000) in terms of active intensive flow on the bag fabric and this causes counter movement of bag infill sand. Also highlighting on the same concept, Recio and Oumeraci (2009 b) represented numerically the wave induced velocity causing uplift in a bag and the outer velocity of the bag was found as 0.9 m/s. On the other hand, Kim et al.(2004) defined the allowable value for a specific size of model bags while the whole structure collapses, as these bags are placed using artificial connection units. Among these three available studies the acquired velocities are dependent on the bag size and wave type; so, exactly there is no information on the incipient velocity ranges which could initiate individual geobag movement from a coastal structure. It should be noted that the definition of 'incipient velocity' within the study reported hereien is the velocity required to cause movement of geobags from the revetment structure; it may therefore be considered 'geobag incipient velocity'.

Table 2: Velocity against the performance of costal geobag structures

Year	Reference	Velocity value	Remark
2000	Pilarczyk	1.5 m/s	Velocity above 1.5 m/s increases internal sand movement results in instability.
2004	Kim *et al.*	1–3 m/s	Applicable experimental velocity on different bag size while they are placed on connection unit.
2009b	Recio and Oumeraci	Wave induced velocity outside of bag = 20 × average velocityinside the bag (Numerical model outcome)	The flow through structure is governed by the voids between geobags.

Table 3: Coefficient of drag and lift forces due to wave induced flow on geobag

<table>
<tr><th>Year</th><th>Reference</th><th>Cd</th><th>Cl</th><th>Re</th><th colspan="2">Comment</th></tr>
<tr><td>2000</td><td>Bezuijen et al.</td><td>1</td><td>–</td><td>–</td><td colspan="2">Field test</td></tr>
<tr><td>2004</td><td>Kim et al.</td><td>1</td><td>–</td><td>–</td><td colspan="2">Laboratory test</td></tr>
<tr><td rowspan="3">2009a</td><td rowspan="3">Recio and Oumeraci</td><td>0.5–3</td><td>0.3–1.2</td><td rowspan="2">$8×10^4$–$1.8×10^5$</td><td>Bottom most bag</td><td rowspan="3">Laboratory test</td></tr>
<tr><td>2.5–9</td><td>0.3–1.2</td><td>Middle bag</td></tr>
<tr><td>4–15</td><td>0.3–1.2</td><td>$8×10^4$–$2×10^5$</td><td>Topmost bag</td></tr>
</table>

FINDINGS ON COSTAL GEOBAG STRUCTURES

Among the major parameters, the physical properties and revetment construction methods have already been intensively studied and only few recent studies covered the mechanical and hydraulic properties. Although the importance of exploring the local scale failure modes in wave flume started in 1979 by Porraz et al., in last three decades only few studies have explored this aspect in more detail (Table 5). Failure modes in geobag structures can be categorised using the observations from physical models noted in Table 5 and as per location in the structure (Table 6). The main reasons for failures are friction, inertia, drag and lift forces. To date acquisition of the coefficient of friction for force calculations is from a direct shear test (Kim, Yoo et al. 2004; Krahn, Blatz et al. 2004), as there is no available standard for determining this value when considering the whole structure. Recio and Oumeraci (Recio and Oumeraci 2009 a; Recio and Oumeraci 2009 b) introduced physical modelling on inertia, drag and lift forces in a wave flume.

As this experimental study concentrated on wave loading and did not link failure mode to underlying hydraulic loading, it can be deduced that there is current lack of knowledge on the performance of geobag revetments in riverbank protection works.

Table 4: Findings on costal geobag structures

Parameters		Findings
Physical property	Fabric	Nonwoven needle – punched geotextile (Heerten, Jackson et al. 2000; Corbet 2005).
	Seam strength	Should be at least 80 – 90% of the tensile strength of the fabric (Gadd 1988; Saathoff, Oumeraci et al. 2007).
	Sand filling ratio	To avoid 'interlocking' problem among bags, the fill ratio of approximately 80% is defined as an optimum stability of the elements (Figure 1).
	Saturation of the fill	The degree of saturation influences the geobag weight and the falling velocity during dropping. A low degree of saturation (i.e. dry sand fill) increases the capacity of the sand to absorb energy during impact on the bottom of the bag (Bezuijen, Groot et al. 2004).
Revetment construction	Slope steepness	With experience from different type layer–to–layer over lapping, such as – face to face (Venis 1968), 50% overlapping (Porraz, Maza et al. 1979; Kobayashi and Jacobs 1985; Gadd 1988; Recio and Oumeraci 2009 a); the optimum setup can be achieve from 50% overlapping.
	Drop test	Irrespective of the initial orientation, laboratory experiments showed geobags sink under water with the largest axis towards stream wise direction if a sufficient water depth is available (Dassanayake and Oumeraci 2010).
	Geobag launching	In the field, bag placement with its longest axis as a function of water depth between 15 m to 22 m, and a standard deviation of less than 1 m can be achieved if the water depth is limited to 10 m (Bezuijen, Groot et al. 2004).
Mechanical property	Interface Friction	The average friction angle between geobags found to be 30° (Figure 3).
	Permeability	The total forces and moments for geobag displacement in a structure depend on the wave pressure propagation inside the internal gaps between bags (Recio and Oumeraci 2009 a).
	Deformation	The infill sand accumulates at the seaward end and leads to the deformation of the latter part of the bag. This reduces the contact areas with the neighbouring bags (Recio and Oumeraci 2009 a)). Then internal movements of the sand are activated by an incremental horizontal displacement of the geobags. Pilarczyk (2000) reported bag rolling initiation due to internal sand movement caused by surrounding flow velocity more than 1.5 m/s.
Hydraulics	Incipient velocity	Geobag becomes unstable above a flow velocity of 1.5 m/s (Pilarczyk 2000).
	Forces	In wave flume experiments, the coefficient of drag and lift forces found as a function of Reynolds numbers and the roughness of geobags (if 10^4 > Re>10^6) (Recio and Oumeraci 2009 a).

Table 5: Published failure mode observation in physical modelling

Year	Authors	Failure modes	Failure reason
1979	Porraz *et al.*	Slide	○ Friction.
		Push	○ Thrust force due to waves.
		Pullout	○ Uplift pressure and wave current.
1985	Kobayashi and Jacobs	Plugging, collapsing and surging	○ Combined effect of slope angle and wave period
1988	Gadd	Dislodgement	○ Wave impact and physical property.
2004	Kim *et al.*	Slide, overturn and pullout	○ Friction.
2006	Jackson *et al.*	Pullout/ dislodgement	○ Physical property of bags; and ○ Geobag–geobag friction.
2007	Saathoff *et al.*	Overtopping	○ Wave run–up and freeboard.
		Uplift	○ Wave run–down.
2009b	Recio and Oumeraci	Slide	○ Bag submerged weight and lift force; ○ Friction.
		Pullout effect	○ Several wave cycles on the structure; ○ Relatively longer experimental time.

Table 6: Published failure location observation in physical modelling

Year	Authors	Failure locations	Failure reason
2008b	Mori *et al.*	Whole structure fail	○ Hydraulic stresses in bags due to wave load.
		Top bags in structure	○ Wave height; and ○ Wave period.
2009b	Recio and Oumeraci	Top bags in structure	○ Wave uprush induces landward uplift and overturning of bag; ○ Uplift, deformation of bag and wave down rush results in seaward overturning; and ○ Sliding.
		Slope bags in structure (bags located just below and below the surface water level)	○ Large wave height; and ○ Sliding.

Thus along with Table 6, field monitoring of coastal geobag structures indicated that overtopping, sliding, puncturing, pullout/dislodgement and toe scour are the most common failure modes (Jackson, Corbett et al. 2006; Mori, Aminti et al. 2008 ; Recio and Oumeraci 2009 a).

➢ **Geobag structures for riverbank**

Geobags have been used in river protection structures since last decades (Oberhagemann and Hossain 2011). To better understand the issues surrounding geobag placement for revetment structures in Jamuna – Meghna River erosion protection scheme,(Oberhagemann and Hossain 2011) used 1:20 scale models in four categories of test (drop, launch, incipient motion, mega container). The majority of these were designed to investigate a range of launching and placement methods. Akter et al (Akter, Pender et al. 2010) have identified pullout/dislodgement, sliding, slumping and physical damage as potential failure mechanisms for geobag in revetments. Similar findings are also noted for the coastal protections so the acquired information suppose to be helpful for understanding the performances of geobags in riverbanks(Table 4):

- Sandbag of nonwoven needle – punched geotextile fabric with seam strength of at least 80 – 90% of the tensile strength of the fabric and approximately 80% fill ratio would provide desirable performances;
- The 50% layer–to–layer over lapping of sandbag achieved optimum revetment stability whereas the average friction angle between sandbag found to be 30°; and
- Geobag becomes unstable from a revetment above a flow velocity of 1.5 m/s.

CONCLUSIONS

Sand filled bags (sandbags) became long term coastal protection while replaced by geotextile as bag fabric. Experiences and researches on geobags specified details on bag materials, construction of geobag structures and hydrodynamic forces. Due to cost efectiveness and easy handling, geobags are recently applying for long term riverbank protection. To contribute in the knowledge field, this study covered all the possible published literatures ranging from sandbags to geobags. This is expected that the performances of geobag revetment in riverbank can be strengthen with this acquired knowledge.

REFERENCES

Akter, A., G. Pender, et al. (2010). Modelling the hydrodynamic forces on geobag revetments using CES. First European Congress of the IAHR. Edinburgh, UK.

Bezuijen, A., M. B. Groot, et al. (2004). Placing Accuracy and Stability of Geocontainers. 3rd Geosynthetics Conference, Munich, Technische Universität München, Cent, Geotech.

Breteler, M. K., K. W. Pilarczyk, et al. (1998). Design of Alternative Revetments. 26th International Conference on Coastal Engineering, Copenhagen, Denmark.

Corbet, S. (2005). Geosynthetics in Construction. United Kingdom, Geotechnical Faber Maunsell Limited.

Dassanayake, D. and H. Oumeraci (2010). Sinking Behaviour and Deformation of Geotextile Sand Containers (GSCs) during construction of fully submerged GSC-structures. International Workshop on Geosynthetics and Modern Materials in Coastal Protection and Related Applications. IIT Madras, Chennai, India.

Gadd, P. E. (1988). Sand Bag Slope Protection: Design, Construction and Performance. Arctic Coastal Processes and Slope Protection Design, American Society of Civil Engineers: 145-165.

Grüne, J., U. Sparboom, et al. (2006). Stability tests of geotextile sand containers for monopile scour protection. 30th International Conference Coastal Engineering (ICCE), San Diego, California, USA.
Gutman, A. L. (1979). Low-cost Shoreline Protection in Massachusetts. Coastal Structures ’79, New York, American Society of Civil Engineers.
Heerten, G., L. A. Jackson, et al. (2000). New developments with mega sand containers of non-woven needle-punched geotextiles for the construction of coastal structures. International Conference on Coastal Engineering 2000. Sydney, Australia.
Heibaum, M. H. (1999). "Coastal scour stabilisation using granular filter in geosynthetic nonwoven containers." Geotextiles and Geomembranes **17**(5-6): 341-352.
Hornsey, W. P., L. A. Jackson, et al. (2003). Large sand filled geotextile containers as a construction aid over poor quality marine clay. Coasts & Ports Australasian Conference. Auckland, New Zealand**:** 1-8.
Jackson, L. A., B. B. Corbett, et al. (2006). Failure modes and stability modelling for design of sand filled geosynthetic units in coastal structures. 30th International Conference on Coastal Engineering. San Diego, USA.
Kim, H. T., D. Yoo, et al. (2004). A fundamental approach for an investigation of behaviour characteristics of the vegetation structures using sandbags. GeoAsia 2004**:** 225-232.
Kobayashi, N. and B. K. Jacobs (1985). Experimental study on sandbag stability. Coastal Zone 85: 4th Symposium on Coastal and Ocean Management, American Society of Civil Engineers.
Korkut, R., E. J. Martinez, et al. (2007). "Geobag Performance as Scour Countermeasure for Bridge Abutments." Journal of Hydraulic Engineering **133**(4): 431-439.
Krahn, T., J. Blatz, et al. (2004). Assessment of sandbag dike interface shear using two direct shear devices. 57th Canadian Geotechnical Conference. Canada.
Mori, E., P. L. Aminti, et al. (2008). Field experiment on a groin system built with sand bags. International Conference on Coastal Engineering. Hamburg, Germany**:** 219.
Oberhagemann, K. and M. M. Hossain (2011). "Geotextile bag revetments for large rivers in Bangladesh." Geotextiles and Geomembranes **29**(4): 402-414.
Pilarczyk, K. W. (2000). Geosynthetics and geosystems in hydraulic and coastal engineering, A.A. Balkema, Rotterdam, Netherlands.
Porraz, M., J. A. Maza, et al. (1979). Mortar-filled Containers, Lab and Ocean Experience. Coastal Structures ’79, American Society of Civil Engineers
Recio, J. and H. Oumeraci (2009 a). "Process based stability formulae for coastal structures made of geotextile sand containers." Coastal Engineering **56**(5-6): 632-658.
Recio, J. and H. Oumeraci (2009 b). "Processes affecting the hydraulic stability of coastal revetments made of geotextile sand containers." Coastal Engineering **56**(3): 260-284.
Restall, S. J., L. A. Jackson, et al. (2002). "Case studies showing the growth and development of geotextile sand containers: an Australian perspective." Geotextiles and Geomembranes **20**(5): 321-342.
Saathoff, F., H. Oumeraci, et al. (2007). "Australian and German experiences on the use of geotextile containers." Geotextiles and Geomembranes **25**(4-5): 251-263.
Venis, W. A. (1968). "Behaviour of dumping material when exposed to current and wave action, Closure of Estuarine Channels in Tidal Regions." De Ingenieur **50**: 159 -167.

DEVELOPMENT OF CYCLONE HAZARD MAPS WITH EFFECT OF CLIMATE CHANGE SCENARIO FOR BARGUNA SARDA UPAZILA, BANGLADESH

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ABSTRACT

Bangladesh is one of the top susceptible countries to the climate change and the adverse impact on the country will be catastrophic because of the convergence of climate change, poverty and large population. The frequency of natural disaster like floods, cyclone etc have increase significantly over the last decade particularity in the costal line of Bangladesh which is affirmed as the impact of climate change. The consequences of these disasters are a huge loss of lives and properties that implicates the economy of the country. So disaster management is now an important concern to minimize all those losses. Disaster management of an event like Cyclone, Flood or Earthquake etc. requires some ingredients, such as, response, incident mapping, establishing priorities, developing action plans and implementing the plan to project lives, properties and the environment. GIS in combination with Remote Sensing (RS), can use effectively to identify hazard and risk of cyclone. In Bangladesh, cyclone and tidal surge are considered as the most catastrophic phenomena for coastal regions. From the historical data, it is seen that during the year 1797 to 2013, Bangladesh has been hit by 66 severe cyclones, 35 of which were accompanied by storm surges. The main focus of the study is to develop a hazard map with and without the effect of climate change that could be used for disaster and management. Hazard mapping of 50 year return period has been developed integrating historical cyclone data with Digital Elevation Model (DEM) and interpolation of storm surge height. Satellite images and field survey data are used in GIS and RS platform to assess the land use pattern and developing land use map. Cyclone hazard mapping of the study area which can be implied to give a signal of awareness to the local community and the decision makers to provide advance planning for cyclone disaster management.

Keywords: *hazard mapping, Interpolation, Land use, Return Period*

1. INTRODUCTION

In Bangladesh cyclone and tidal surge are consider the most catastrophic phenomena of coastal regions. Most of the coastal areas of the world are at risk from natural hazards resulting from geological or meteorological disturbances. The coastal region of Bangladesh which is formed out of the process of sedimentation by the mighty river systems of the Ganges, the Bramaputra, and the Meghna, particularly vulnerable to cyclonic storm surge floods due to its location in the path of tropical cyclones, wide and shallow continental shelf and the funneling shape of the coast(Das,1972). Cyclone-induced storm surges in this region typically originate in the central and southern parts of the Bay of Bangal or in the Andaman Sea. Due to the shallow continental shelf, the surge amplifies to a considerable extent as it approaches land and causes disastrous floods along the most coast(Murty et al,1986). In the northan of Bay of Bangal, a unique combination of high tides, a funneling coast configuration, the low flat coastal terrain and high population density have produced some of the highest mortality figure associated with storm surge(Flier and Robinson,1972). All the ingredients for a major cyclone disaster and such disasters have occur several times in the past and claimed hundreds of lives notably in 1991 and 2013. Rather Bangladesh is one of the top vulnerable countries to the climate change. Due to climate change and sea level raise , the country is likely to be affected by more intense cyclone events in the foreseeable future. The consequences of climate change lead to an increase in the cyclone-prone area and put a large number of people at risk.however, over the last decade the country's capacity to deal with cyclones has improved considerably with the establishment of a cyclone warning and evacuation system. For example, cyclone Sidr in 2007 claimed far fewer lives (approx. 3500) than the 1970 and 1991 cyclones, which killed at least 500000 and 138000 people respectively. At present , people in the flood risk areas are directed by the forecasting and warning centre to evaluate the refugee shelters. Unfortunately, the capacity of existing shelters is not adequate to accommodate all the people in the flood risk areas.

More recently, advances in computer, GIS and RS technology have increase the accessibility and mobility of GIS tools. As a consequence, GIS has now become a fundamental component of community-based methodologies. Thus, integrating local knowledge with GIS and RS techniques in the disaster identification stare in order to develop map and access the hazard prone areas is an excellent tools for cyclone disaster management.

2. METHODOLOGY AND DATA

Satellite Images with the integration of Geographical Information System (GIS) are used for historic storm surge flood hazard analysis.GIS has widely been used to map and model surface water and flood hazard (Aziz et al, 1998 (as cited in (Dewan, A.M. 2004)). Digital Elevation Model (DEM) based storm surge flood extent with depth, an integral part of GIS can be adopted for cyclone hazard study. To get storm surge flood map of a study area, flood elevation generated from water level data, is subtracted from ground elevation data (Dewan, et al., 2004). The main disadvantage of the method is unavailability of interpolated water level surface during subtraction with land elevation surface which is interpolated. For obtaining flood extent it is necessary to have both interpolated water level and land elevation surfaces as flooding is a continuous phenomenon so that and interpolation is the procedure of estimating the value of properties at unsampled points or areas using a limited number of sampled observations.

In order to resolve the methodological gap, interpolation technique at GIS system has been applied using water level data of different points in order to generate interpolated water level surface generation. There are number of interpolation techniques, designed for particular purpose are available in ArcGIS framework. One of is Kriging interpolation, which has been developed based on statistical models that include autocorrelation (ESRI, 2007). But for water level surface generation, the technique will not be appropriate as there is no statistical relation between the different stations in real scenario. On other hand, another interpolation technique, Spline is used for land surfaces generation, as the technique estimates values using a mathematical function that minimizes overall surface curvature, resulting in a smooth surface that passes exactly through the input points (ESRI, 2007). But for interpolating a hydrologically correct surface only Topo to Raster method available in ArcGIS 9.3.1 is used (ESRI, 2007). In the present study, for inundation mapping from point elevation data sources Topo to raster interpolation tool of ArcGIS has been applied for generating water level interpolated surface. The point feature datasets can be converted to 1m resolution ArcGIS grid format datasets using the Topo to Raster tool located in the ArcGIS Toolbox (Tait, et al., 2007). The Topo to Raster tool in ArcGIS 3D analyst results in a connected drainage structure and corrects representation of ridges and streams (Collins, Eric, Forkuo and Mensa, 2012). From field surveys data and Study conducted by (Tamima, 2009) showed, surge height level at Patharghata, near to Barguna Sadar upazila was in the range of 5.5 to 6.0 m (PWD) for the cyclone SIDR. In this connection, the mentioned surge height has been used for Topo to Ras interpolation technique to obtain the interpolated water level surface in the study area during SIDR. The interpolation method is specifically designed for the creation of hydrologically correct DEM (ESRI, 2007).

Based on the bay configuration, tidal amplitudes and bathymetry, the entire coastal belt is divided, into three zones and our study area falls in to zone-1 (khan,1995). Bangladesh meteorological department historical data are taken into consideration for frequency analysis and result show that the corresponding wind speed for 5,10,20 and 50 are of 166, 205, 243 and 291 km/h respectively. Historical cyclone wind speeds and surge heights of zone-1 are used in the regression analysis result, storm surge heights are showed using the frequency analysis result for 5, 10, 20 and 50 are of 3.1, 3.9, 4.7 and 5.7 meters respectively as calculated by (Rana, 2010). According to the study conducted by (Rana, 2010), cyclone SIDR is a 50 year return period hazard event. Another interpolation technique, Spline can also be used for surfaces generation as the technique estimates values using a mathematical function that minimizes overall surface curvature, resulting in a smooth surface that passes exactly through the input points (ESRI, 2007). The study area surface behaves as almost flat surface. Barguna, Sadar Upazila, located in the Ganges- Brahmaputra delta is almost plain land where the elevation ranges from 15m in the north to nearly a meter in the south and the gradient of the delta surface is about 0.016m/km (Banglapedia, 2008).

An essential data required for hydraulic simulation and/or GIS interpolation of water surface is land topography. The NASA Shuttle Radar Topographic Mission (SRTM) digital elevation data(DEMs) were downloaded from the SRTM FTP server (ftp://e0srp01u.ecs.nasa.gov/srtm/version2/) for the study area. The SRTM data is available as 3 arc second (approx. 90 m resolution) DEMs. The DEM data were further processed using ArcGIS 9.3 to fill in the no-data voids or cells. The processing involved the production of vector contours and points, and the re-interpolation of these derived contours back into a raster DEM, following by filling in no-data cells using Spatial Analyst. The data were then projected to WGS84 projection.

Difference between water level data obtained from interpolated water level surface during SIDR and land surface values extracted from SRTM has been considered as inundation depth in the study area. Land use or land cover dataset has been generated from the digital image classification of LANDSAT, ETM+ satellite images (Samanta, et al., 2011). Supervised image classification method has been adopted on LANDSAT 7 image downloaded from Global Land Cover Facility web site (Alaguraja, et al., 2010). In this present study, Landsat satellite image has also been used for land use map generation with the help of Integrated Land and Water Information System (ILWIS 3.4) software. Here, only agricultural, rural settlement, forest coverage have been identified. For water bodies identification Google earth image digitizing process has been used. It is necessary to note that, as like as (Dewan, et al., 2004), rainfall induced flooding, and / or water logging, inside the study area has not been considered during preparation of flood depths maps. Finally, Inundation layers have been overlaid on landuse layer to obtain the overlaid zones termed as storm surge flood hazard zones between them.

On the approach of development of hazard map with the effect of climate change, B1 scenario has been consider among six emission scenarios. According the study conducted by (IWM, CEGIS, 2007), shows 23 cm sea level rise would be occur in 2050. So, based on the study findings inundation map has been developed . Form the ArcGIS analysis different sort of inundation statistics have also been generated with and without the effect of climate change.

3. STUDY AREA

Barguna district is located in the estuarine environment of the Payra and the Bishkhali River with the Bay of Bengal confluence. This study area centrifuged at Barguna Sadar Upazila falls under SC-55 hydrologic region of Bangladesh and which is about 454.39 sq.km (Banglapedia, 2008). The study area is bounded by the Bishkhali River at west, The Payra River at east, The Bey of Bengal at south. About 49% of the study area population is landless which is 5% less from coastal statistics. Percentage of marginal farmer is 57% which is 4% more from national figure. The agro-climatic condition with fertile soil gives the study area a rich potential for diversified agriculture and other economic activities but the salinity in irrigation water suffers from agriculture extremely.

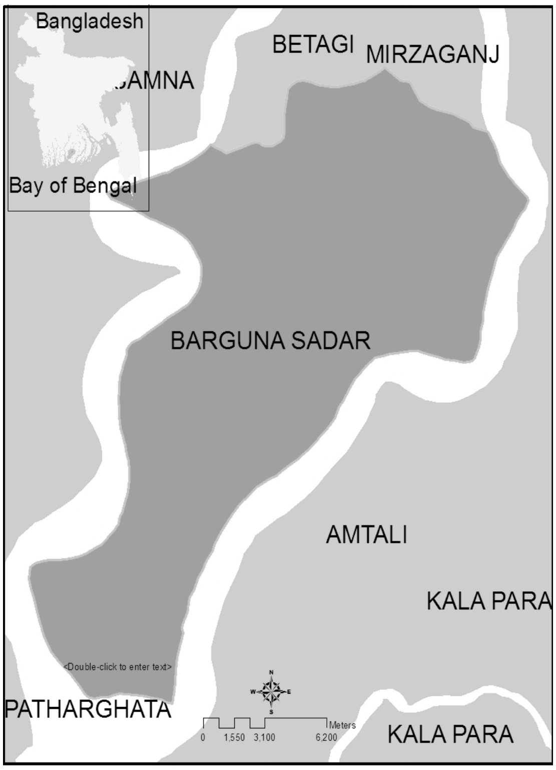

Figure 7: Study area

4. DEVELOPMENT OF LAND USE MAP

The land in the study area is mostly agricultural. The land use of the area comprises agriculture, forest, fisheries, wetlands and urban settlements. The statistics of the major land coverage has been given in Table 1. The land coverage map of the study area has been presented (Figure 2).

Table 1: Statistics of the major land coverage of Barguna Sadar Upazila

Land use	Area (sq.km)
Cropping Land	223.99
Forest	4.27
Rural Settlement	76.17
Water bodies	59.27
Others use	95.67

In the study area, rice is the most important food crop whenever Aman and Boro (Rabi) crops are the traditionally dominant cropping pattern of the study area. Other major crops include sugarcane, oilseeds, lentil, potato, sweet potato, jute, betel leaf, sesame etc. In Kharif I (March- June) local broadcast Aus and broadcast Aman are the dominant crops with minor crops of oilseeds and fruits. In Kharif II (July-October) broadcast Aman continues to grow in lowlands. Local transplant Aman and high yield variety (HYV) transplant Aman are grown mostly in high lands. In the Rabi season (November-February), Lentil, Mustard, Mashkalai, Gram, Chilli, Brinjal, Bean, Borboti, Sesame, Onion, Garlic, Sugarcane, etc. are planted in addition to HYV Boro. Fruits like banana, papaya, and guava are grown year-round.

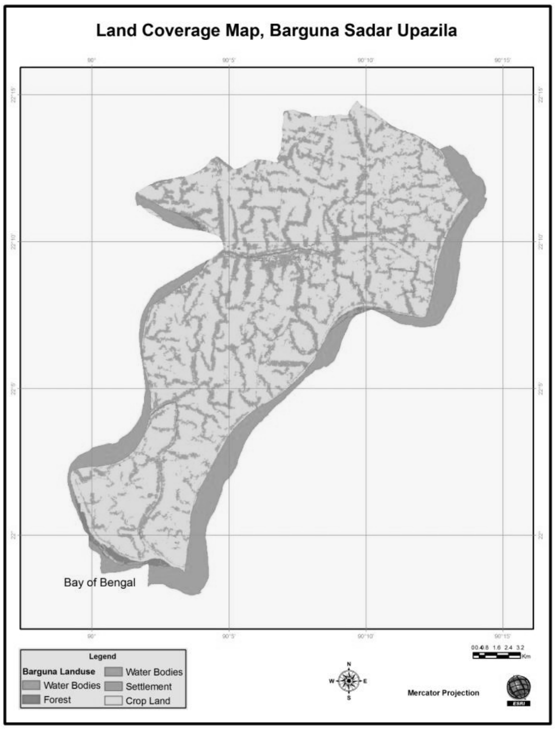

Figure 2: Land Coverage Map of Barguna Sadar Upazila

5. DEVELOPMENT OF SLOPE MAP

Major part of Study area lies in the lower Meghna River flood plain and the estuarine environment of Bay of Bengal. The lands of the district is almost flat having mild slope (0.40 –1.14 degree) from north to south-east (Figure 5.16). The surface elevation profiles also show that the study area is slightly sloping both in North-South (0.02O) and East- West (0.06O) direction. The study area is located under the physiographic unit called Ganges Tidal Floodplain (Banglapedia, 2008). The sediments are mainly non-calcareous clays, but they are silty and slightly calcareous on riverbanks and in a transitional zone in the east adjoining the lower Meghna (Banglapedia, 2008).

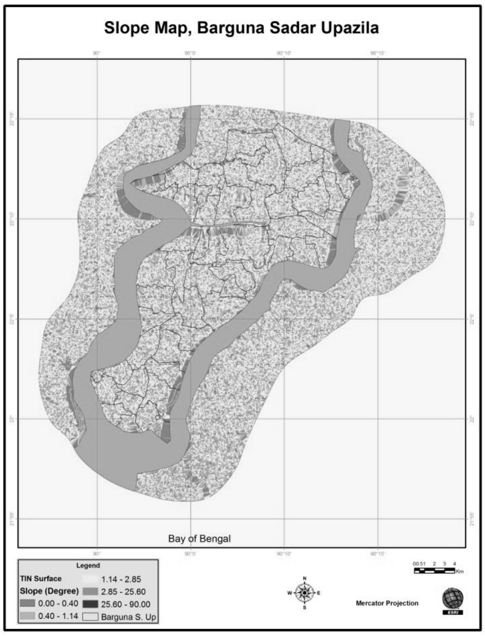

Figure 3: Slope Map of Barguna Sadar Upazila

6. DEVELOPMENT OF HAZARD MAP

In this study, for 2007(Sidr), 54.00 (249 Sq. Km) percent area will be flooded. Among them Agriculture and settlement area are 31.04 and 10.60 percent respectively. In 5.7 meter surge height 92.96 percent road will be inundated.

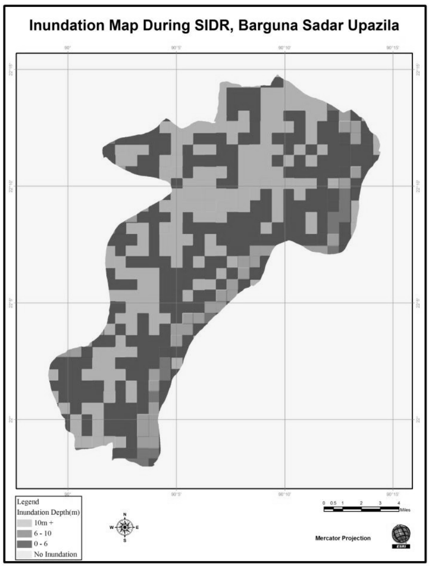

Figure 4: Hazard Map of 50 yr return period

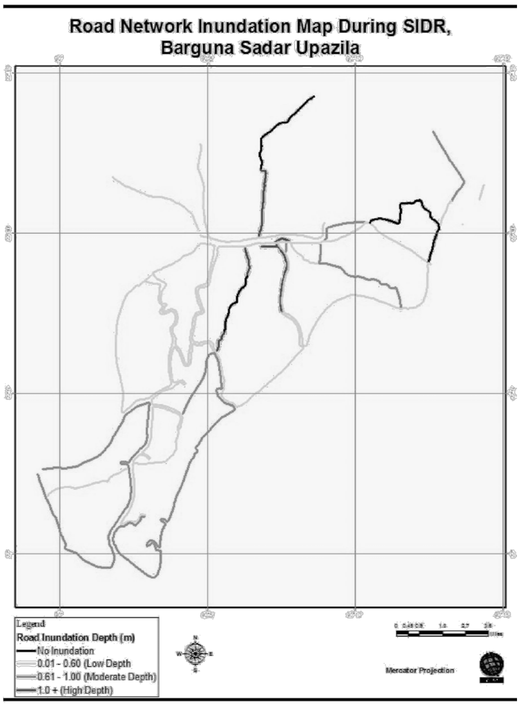

Figure 5: Road Inundation Map During, 2007(Sidr)

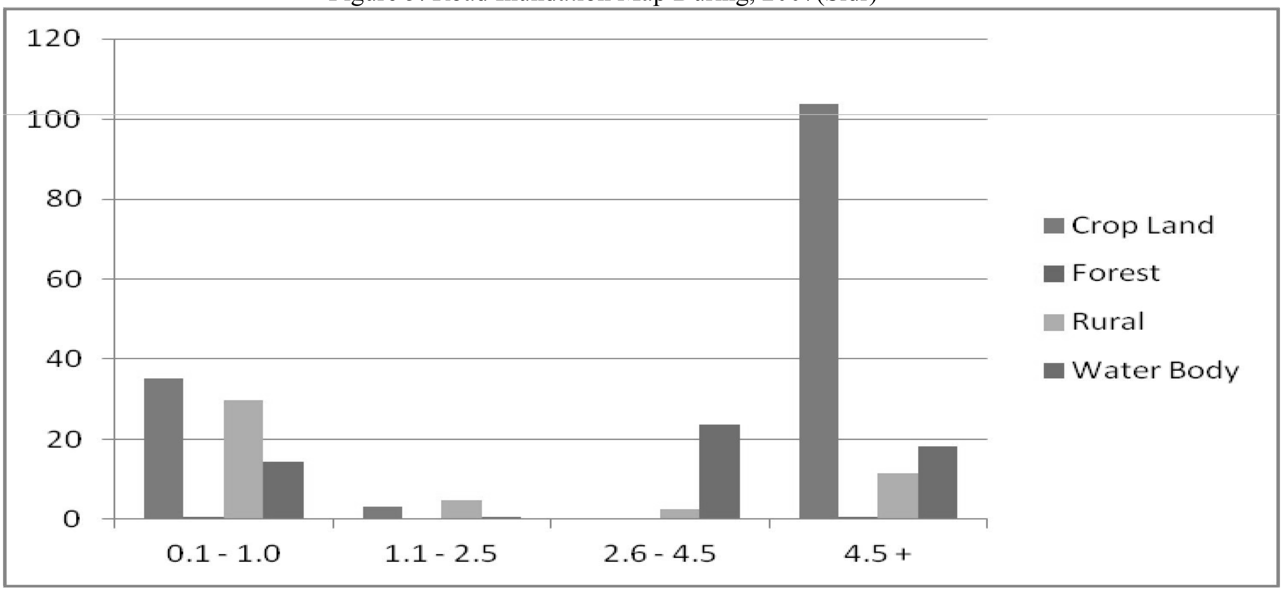

Figure 6: Inundations and Adjacent Land Coverage, 2007(Sidr)

7. ILLUSTRATIONS CLIMATE CHANGE SCENARIO FOR COASTAL STORM SURGE

Climate change is the greatest environmental challenge facing the world today. The threat of sea level rise spans an enormous range of possible impacts from the relatively small and manageable to the catastrophic. Bangladesh has been identified as one of the most vulnerable countries to the impacts of global warming induced accelerated sea level rise. Taking the greenhouse gas-emission scenarios from 3rd IPCC, it is estimated that the global rise in sea level from 1990 to 2100 would be between 9 and 88 cm. Global sea level rise for the projected year 2020, 2050 and 2080 has been selected from Third Assessment Report (TAR) of IPCC 2001 for high and low emission scenarios of SRES (Special Report on Emission Scenarios) having four families, A1, A2, B1, B2 and six emission scenarios. As the present study chose the B1 scenario and according the study conducted (IWM, CEGIS, 2007) 23 cm sea level rise would be occur in 2050. So, based on the study findings following inundation map has been obtained (Figure 7).

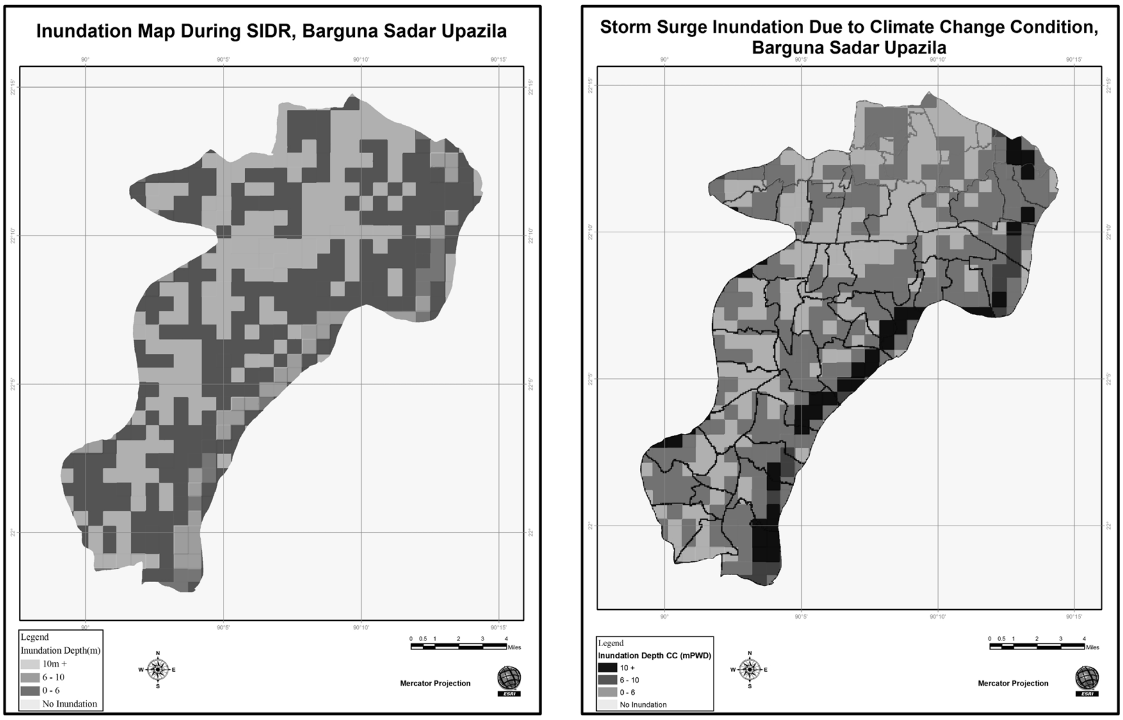

Figure 7: Inundated Area with and without Climate Change Scenario in Barguna Sadar Upazila, 2050

Table 2 Inundation Statistics with and without Climate Change Scenario in Barguna Sadar Upazila, 2050

Inundation Extent	Crop land	Rural Settlement	Water Bodies	Others
Without Climate Change (sq.km) Total (249)	142.29	48.65	57	1.06
With Climate Change (sq.km) Total (250.69)	143.56	48.89	57	1.24
Percentage Increase (0.68)	0.89	0.49	0	16.98

8. CONCLUSIONS

The present research could be a good and an effective methodological study in storm surge flood hazard study whereas, the application of remote sensing and GIS is a example of disaster management tools. Moreover, it can be said that, in order to manage floods in the study area, disaster management practices should be designed on the basis of result findings. In addition, land use policy might be adopted on basis of the study result. Special attention should be drawn for storm surge flood management of the coastal area. So, the planners and the decision makers may find the result of the study useful for framing an appropriate disaster risk management plan in the Barguna Sadar Upazila.

ACKNOWLEDGEMENTS

I wish to express my deepest thanks to IWM and CEGIS authorities for providing me important papers, GIS data and other technical matters related to my study.

REFERENCES

Alaguraja, P. et al., 2010. Land Use and Land Cover Mapping – Madurai District, Tamilnadu, India Using Remote Sensing and GIS Techniques. International Journal of Civil and Structural Engineering, 1(1), pp. 91 - 100.

Banglapedia, 2008. Dhaka: Bangladesh Asiatic Society.

Collins, Eric, Forkuo and Mensa, 2012. River Inundation and Hazard Mapping – a Case Study of Susan River – Kumasi. s.l., GSDIA.

Das, P.K., 1972. A prediction model for storm surges in the Bay of Bengal, Nature 239, 211-213.

Dewan, A., Nishigaki, M. & Komatsq, M., 2004. DEM Based Flood Extent Delineation in Dhaka City, Bangladesh. Journal of the Faculty of Environmental Science and Technology, 9(1), pp. 99 - 110.

ESRI, 2007. Introduction to ArcGIS Spatial Analyst. s.l.:Environmental System Research Institute.

Flierl, G.R., and A.R. Robinson, 1972. Deadly surges in the Bay of Bengal: Dynamic and storm tide tables. Nature, 239, 213-215

Institute of Water Modeling, 2010. Report on Flood Hazard Model for Sirajganj District, Dhaka, IWM.

IWM, CEGIS, 2007. Investigation the impact of relative sea-level rise on coastal communities and their livelihoods in Bangladesh.

Khan, S.R., 1995. Geomorphic characterization of cyclone hazards along the Coast of Bangladesh. M.Sc. Thesis, ITC, Enschede, the Netherlands.

Murty, T.S., Flather, R.A., Henry, R.F., 1986. The storm surge problem in the Bay of Bengal. Progress in Oceanography 16, 195-233.

Rana, M.S.,Gunasekara, k., Hazarika, M.K., Samarakoon, L. & Siddiquee, M., 2010. Application of remote sensing and GIS for cyclone disaster management in coastal area : a case study at Barguna District,Bangladesh. International archives of the photogrammetry, remote sensing and special information science.

Samanta, S., Pal, D., Lohar, D. & Pal, B., 2011. Preparation of Digital Data Sets on Land use/land cover, Soil and Digital Elevation Model for Temperature. Indian Journal of Science and Technology, 4(6), pp. 636 - 642.

Tait, Sturman & Smart, 2007. Inundation mapping of future high tides, sea level rise and storm surge.

Tamima, U., 2010. population evacuation need assessment in cyclone affected Barguna District. Journal of Bangladesh institute of planners, 2, 145-157.

RESPONSE OF PROTECTIVE VEGETATION DUE TO WAVE LOADING

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ABSTRACT:

The coastal areas around the Bay of Bengal are vulnerable to strong winds and storm surges. With the increased effects of climate change, the threat of cyclonic storms poses an even larger threat in this disaster-prone country. To minimize the damage and loss, it is required to build safe structure. But most people in the coastal areas cannot afford to build strong structures, which can sustain in the extreme conditions. Coastal Protective vegetation can be used to reduce the magnitude of losses. Measurement of wave forces and modeling of fluid dynamics suggest that tree vegetation may shield coastlines from such damage by reducing wave amplitude and energy. The main objective of the present work is to explore the effect of ocean wave loads on the dynamic response of protective vegetation along the coastline. Two tree models (of diameters 10″and 20″for heights 20′and 50′respectively) are subjected to hydrodynamic wave load from design storms in coastal areas according to BNBC. Nonlinear dynamic analysis of protective vegetation shows the significant effect of storm surge and structural height on deflection and curvature; however, the total energy loss encountered in the process significantly reduces the storm intensity and the vegetation is thus found to provide significant protection for the coastal structures.

Keywords:*Natural Hazards, Wave, Cyclone, Protective Vegetation, Nonlinear Dynamic Analysis.*

1. INTRODUCTION

Bangladesh is situated at the interface of two contrasting settings with the Bay of Bengal and the North Indian Ocean to the south and the Himalayas to the north. The geographical location, low and almost flat topography, very high population density, etc. have made this country one of the most vulnerable countries of the world to be affected by the impact of climate change. The funnel-shaped northern portion of the Bay of Bengal causes tidal bores when cyclones make landfall and thousands of people living in the coastal areas are affected. Of the 508 cyclones that have originated in the Bay of Bengal in the last 100 years, 17 percent have hit Bangladesh, amounting to a severe cyclone almost once every three years (GoB, 2008). From the statistical analysis of the recorded cyclones over the last 200 years, it has been found that number of occurrences of major cyclones has drastically increased in the recent decades. While the number of cyclones was 3 during the period of 1795-1845 and 1846- 1896 respectively, the number increased to 13 during 1897-1947 and 51 during the period of 1848-1998.

More than three million people live in high-risk areas along the 400-km coastline. The deadly cyclone of 1991 killed about 138,000 people, injured about the same number and left more than 300,000 homeless, bringing the count of those affected by the disaster to about 15.5 million, with an estimated damage of about $2 billion. Following this, the government and several international NGOs initiated a disaster preparedness and management program, including construction of cyclone shelters in vulnerable coastal areas. Today, a disaster warning system and evacuation procedure are in place, alongside more than 2500 multi-storied concrete cyclone shelters along the coastline. As a result, in the severe cyclone of 1997, the number killed and injured were 111 and about 10,000 respectively, demonstrating a great improvement over the 1991 figures. However, it had still left about one million people homeless. The figures from the more recent cyclone Sidr (2007) and Aila (2009) were even worse, both in terms of loss of lives, injuries as well as property damage.

The country has three distinct coastal regions, namely the western, central and eastern regions. The western zone is very flat and low and is crises-crossed by numerous rivers and channels. The central region is the most active one and continuous process of accretion and erosion is going on there. The eastern region is covered by hilly areas and it is more stable and has a long beach there. Some parts of western region have the capacity to stand against cyclone disaster where Sundarban, the largest mangrove forest exists. But the other parts of the coastal area have no significant protective barrier to dissipate the cyclone and tsunami energy.

Mitigation, preparedness, and escape are representative countermeasures to protect humanlives and infrastructure facilities from natural hazards in the coastal regions. Mitigation techniques are broadlycategorized in two ways. These are artificial methods (hard solutions) and natural methods(soft solutions utilizing a natural buffer zone of coastal vegetation, sand dunes, lagoons, orcoral reefs). Artificial methods are mainly kinds of sea walls (huge embankments, tsunamigates) that can be constructed on the coastal area for any predicted storm surge height.However, the construction costs of the artificial methods can be very high, which restrictsdevelopment in many cases. Some developed countries like Japan, which has frequenttsunamis or storm surge threats, have employed such techniques. On the other hand, the useof coastal vegetation as a natural method for disaster mitigation was also discussed in Japanmore than 100 years ago (Honda, 1898). So, this natural method of protective vegetation could be a good solution in the coastal areas of Bangladesh. But, there had not been done any full scale study on the effectiveness of protective vegetation in Bangladesh. However, the reduction of waves and current velocity on passing through mangroves, and other vegetation has been studied thoroughly by many authors. These studies, however, focus on wind-generated waves and tidal inflows. Although they provide important information relevant to the mitigating effect of coastal forests on tsunamis, their results cannot be directly applied, as tsunamis are transient waves with much longer wavelengths, and as such that the impact is much greater when they strike coastal areas.

The main objective of the present work is to implement and study the effect of hydrodynamic wave loading derived from cyclones in the trees of coastal areas; i.e., to study the structural behavior of different types of trees to ocean wave loading.The more specific objective is to study the effect of vegetation in resisting ocean waves.

2. CYCLONES AND VEGETATION

2.1 Cyclones in Bangladesh

A cyclone is one of a family of tropical storms (also called hurricanes, typhoons, or whirlwinds) that develop over warm tropical oceans and have sustained winds of 64 knots (74 mph or 120 kmph). Cyclones develop over warm oceans that are over 27ºC in temperature. Water evaporating from the sea acts as a kind of 'fuel', producing the energy of a cyclone. Not only are the winds dangerous but they also blow on the water, creating the problem of storm surges and huge waves. Water can rise as high as thirty feet and floods can occur up to 30 miles inland.

Table 1: Categories of Cyclone

Category	Wind Speed (kmph)	Sustained Wind speed (kmph) (3 seconds gust)
1	120~155	145~185
2	156~180	186~225
3	181~210	226~265
4	211~250	266~315
5	Greater than 250	Greater than 315

Table 2: Past Devastating Cyclones in Bangladesh

Date	Year	Max. Wind Speed (Km/hr)	Storm Surge Ht. (m)	Deaths
09 Oct	1960	162	3	3,000
30 Oct	1960	210	4.5~6	5,149
09 May	1961	146	2.5~3	11,466
28 May	1963	203	4~5	11,520
11 May	1965	162	4	19,279
12 Nov	1970	223	6~10	5,00,000
25 May	1985	154	3~5	11,069
29 April	1991	225	6~8	1,38,000
15 Nov	2007	240	5~6	3,406
25 May	2009	120	2~3	330

Wind Velocity (Km/h)	Storm Surge Height (m)
85	1.5
115	2.5
135	3
165	3.5
195	4.8
225	6
235	6.5
260	7.8

Table 3: Typical Storm Surge Height for Cyclones in Bangladesh

2.2 Protective Vegetation in Coastal Belt

Various studies show that mangrove forest and other coastal vegetation of certain density can reduce wave height considerably and protect the coast from erosion, as well as effectively prevent coastal sand dune movement during strong winds. Healthy coastal forests such as mangroves and salt marshes can serve as a coastal defense system where they grow in equilibrium with erosion and accretion processes generated by waves, winds and other natural actions. The coastal areas around the Bay of Bengal are vulnerable to strong winds, storm surges, tectonic movement, over-sedimentation, rapid coastal erosion, fluctuating water and soil salinity and long periods of constant flooding.

Based on these field studies, the wave and current characteristics of propagation through the mangrove forest area are as follows:

* The wave height of the swell increases with increasing tidal level, and decreases with increasing proximity to the coast, which suggests wave energy loss caused by bottom friction and resistance to flow by the mangrove vegetation.
* Wave size decreases considerably through denser mangrove areas; therefore, in well-grown and healthy mangrove areas, the effects on wave reduction do not decrease with increasing water depth, which has important practical implications.
* According to the research, the effectiveness of mangroves with *Kandeliacandel* of sufficient height (three to four years old) in reducing wave height per 100 meters was as high as 20 percent and increased to 95 percent when the trees were six years old. At this age, 1-meter wave height on the open coast will be reduced to 0.05 meter at the coast compared to 0.75 meter without mangroves. Vegetation height and density and the width of the area to be planted are important factors in reducing wave height and protecting the coast from erosion. The effect of wave reduction was considerable even when water depth increased due to the high density of vegetation.

Fig. 3.6 is a schematic diagram of wave attenuation model by mangrove forest, while Figs. 3.7(a) and 3.7(b) show the spatial variation of tsunami and cyclone water depth and current velocity due to vegetation as well as higher soils or land. Figs. 3.9(a) and 3.9(b) show the important role of coastal forest and vegetation to diminish the wave height of a 6 m tsunami to 1.6m, and consequently saving coastal structures at West Java

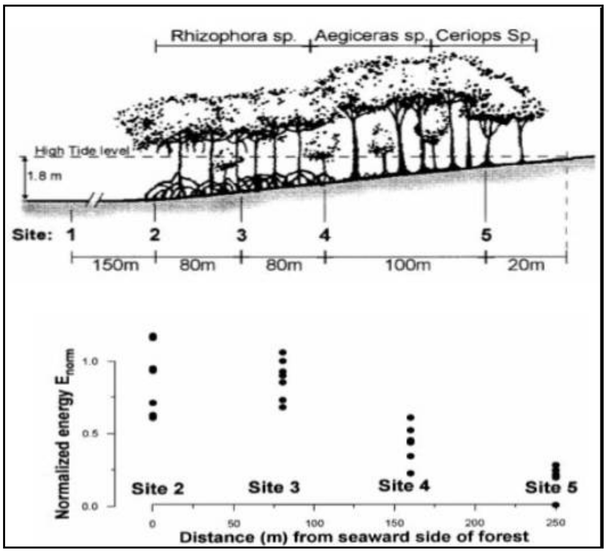

Fig. 1: Wave attenuation by mangrove forest

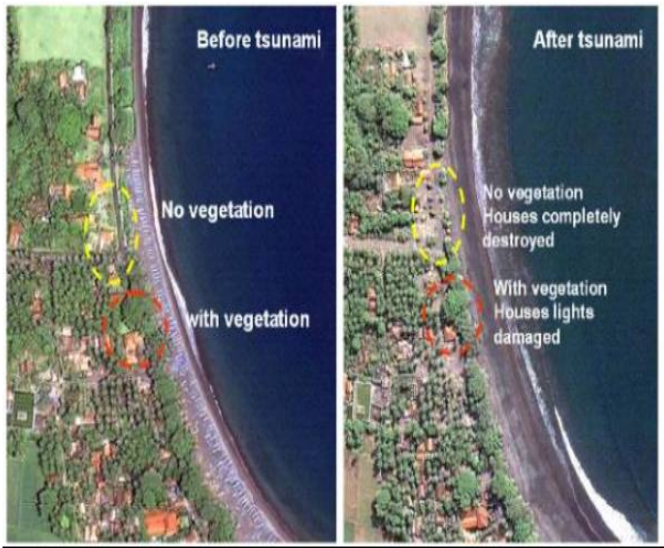

Fig. 2: Coastal forest and vegetation prevented destruction of houses at West Java

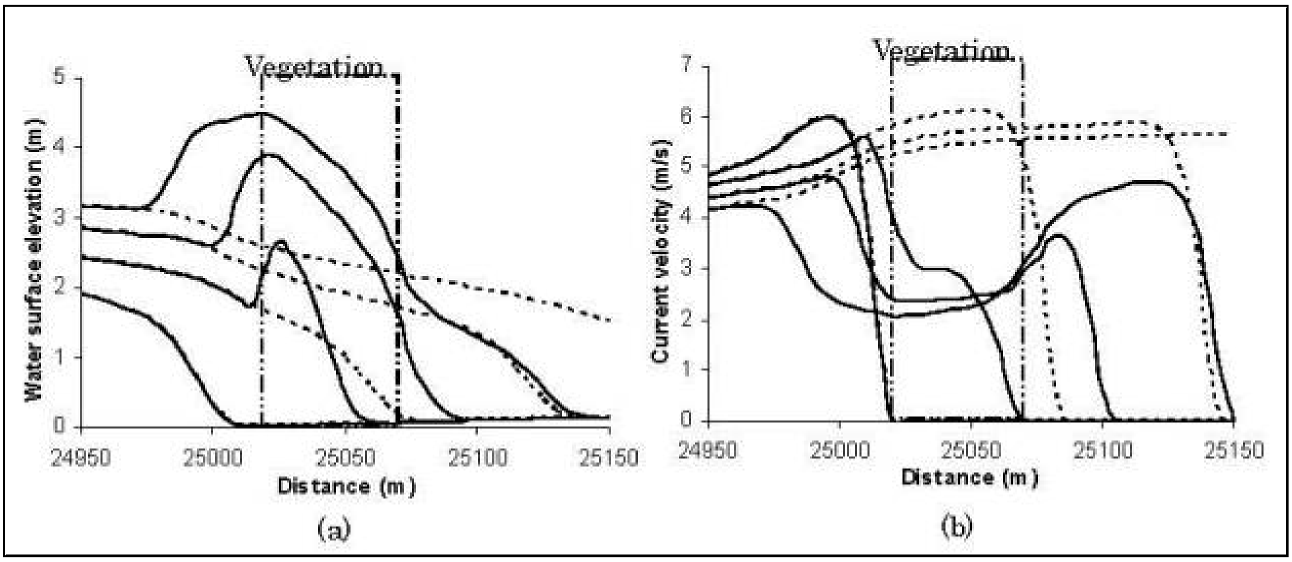

Fig. 3: Spatial variation of (a) water depth, and (b) current velocity for tsunami periods of 10 min [modified by Tanaka from Nandasena et al.(2008b)]. Line: With vegetation, Dashed-line: Without vegetation. Flow is from left to right.

3. LOADS ON COASTAL STRUCTURES

3.1 Wave Loading

Waves can affect coastal structures in a number of ways. The most severe damage is caused by breaking waves. The force created by waves breaking against a vertical surface is often ten or more times higher than the force created by high winds during a storm event. Waves are particularly damaging due to their cyclic nature and resulting repetitive loading. Because typical wave periods during hurricanes range from about 6 to 12 seconds, a structure can be exposed to 300 to 600 waves per hour, resulting in possibly several thousand load cycles over the duration of the storm.

3.2 Wave Spectra

Various idealized spectra are used in oceanography and ocean engineering. The simplest one was proposed by Pierson and Moskowitz in 1964. They assumed that if the wind blew steadily for a long time over a large area, the waves would come into equilibrium with the wind.
The Pierson-Moskowitz equation is

$$S(\omega) = (\alpha\, g^2/\omega^5) \exp\{-\beta(\omega_0/\omega)^4\} \quad \text{……………(1)}$$

where, $\omega = 2\pi f$, f = wave-frequency in Hertz, $\alpha = 8.1 \times 10^{-3}$, $\beta = 0.74$, $\omega_0 = g/\, U_{19.5}$ where $U_{19.5}$ is the wind speed at a height of 19.5m above the sea surface, the height of the anemometers. For most air flow over the sea the atmospheric boundary layer has nearly neutral stability, and

$$U_{19.5} \cong 1.026\, U_{10} \quad \text{……………(2)}$$

assuming a drag coefficient of 1.3×10^{-3}. The frequency of the peak of the Pierson-Moskowitz spectrum is calculated to be

$$\omega_p = 0.877 g/\, U_{19.5} \quad \text{…………(3)}$$

The significant wave-height is calculated to be

$$H_{1/3} = 0.21\, (U_{19.5})^2/g \quad \text{…………(4)}$$

After analyzing data collected during the Joint North Sea Wave Observation Project (JONSWAP), it was found that the wave spectrum is never fully developed. It continues to develop through nonlinear, wave-wave interactions even for very long times and distances. Hence an extra and somewhat artificial factor was added to the Pierson-Moskowitz spectrum in order to improve the fit to their measurements. The JONSWAP spectrum is thus a Pierson-Moskowitz spectrum multiplied by an extra peak enhancement factor γ^r

$$S(\omega) = (\alpha\, g^2/\omega^5) \exp\{-5/4(\omega_p/\omega)^4\}\, \gamma^r \quad \text{……………(5)}$$

where $r = \exp[-(\omega^2 - \omega_p)^2/2\sigma^2\, \omega_p^2]$

3.2.1 Morison's Equation

Morison's equation is easily the most used equation for calculation of hydrodynamic force on marine/offshore structures. For slender structures, it is often considered as valid as the more rigorous diffraction theory, and has been used in several practical as well as research works.

According to the (modified) Morison's equation, the horizontal wave-force and moment on a differential vertical segment *dz*are given (after adjustments for structural acceleration) by

$$dF_x = [K_I a_x + K_D |u_r| u + K_m u_r \partial w/\partial z]\, dz \text{ ; and } dM_{y0} = z\, dF_x \quad \text{………………...(6)}$$

where$K_I = \rho C_I A$, $K_D = \rho C_D R$, $K_m = \rho C_m A$,
[ρ = Water density, C_I = Inertia coefficient, C_D = Drag coefficient, $C_m = C_I - 1$,
A = Cross-sectional area, R = Radius = Half-width of projected surface]
a_x = Horizontal wave-acceleration = $du/dt = \partial u/\partial t + u\, \partial u/\partial x + w\, \partial u/\partial z$,
u_r = Relative horizontal velocity = Horizontal (wave – structural) velocity.

In Eq. (13), the first term gives the inertia force, the second is the drag and the third term corresponds to the axial-divergence term. Total horizontal force $F_x = \int dF_x$and moment about bottom of the structure is $M_{y0} = \int z\, dF_x$, where $\int$ implies integration between $z = -L$ (bottom of the structure) and $z = \eta$ (instantaneous wave-elevation).

3.3 Flood Loads on Structures at Inland Areas

For structures sited at inland areas subject to flood, loads due to flood shall be determined considering hydrostatic effects which shall be calculated based on the flood elevation of 50-year return period. For riverside structures, hydrodynamic forces, arising due to approaching wind-generated waves shall also be determined in addition to the hydrostatic load on them. In this case, the amplitude of such wind-induced water waves shall be obtained from site-specific data.

3.4 Flood and Surge Loads on Structures at Coastal Areas

For structures sited at coastal areas, the hydrostatic and hydrodynamic loads shall be determined as follows

3.4.1 Hydrostatic Loads

The hydrostatic loads on structural elements and foundations shall be determined based on the maximum static height of water, H_m produced by floods or surges as given by

$$H_m = \text{Max}(h_s, h_f) \qquad \ldots\ldots\ldots\ldots(7)$$

where $h_f = y_T - y_g$

h_s = Maximum surge height as specified in (i) below

y_T = Elevation of the extreme surface water level corresponding to a T-year return period specified in (ii) below, meters

y_g = Elevation of ground level at site, meters.

(i) Maximum Surge Height, h_s: The maximum surge height, h_s, associated with cyclones, shall be that corresponding to a 50-year or a 100-year return period as may be applicable, based on site specific analysis. In the absence of a more rigorous site-specific analysis, the following relation may be used:

$$h_s = h_T - (x - 1)\,k \qquad \ldots\ldots\ldots\ldots\ldots(8)$$

where h_T = Design surge height corresponding to a return period of T-years at sea coast, in meters, (i.e., Table 6.2.28 of BNBC'93).

x = Distance (km) of the structure site measured from the spring tide high-water limit on the sea coast, in km; x = 1, if $x < 1$.

k = Rate of decrease in surge height in m/km; the value of k may be taken as 1/2 for Chittagong-Cox's Bazar-Teknaf coast and as 1/3 for other coastal areas

(ii) Extreme Surface Water Level, y_T: The elevation of the extreme surface water level, y_T for a site during monsoon, which may not be associated with a cyclonic storm surge, shall be that obtained from a site-specific analysis corresponding to a 50-year or a 100-year return period.

3.4.2 Hydrodynamic Loads

The hydrodynamic load applied on a structural element due to wind-induced local waves of water, shall be determined by a rational analysis using an established method and based on site-specific data. In the absence of a site-specific data the amplitude of the local wave, to be used in the rational analysis, shall be taken as $h_w = h_s/4 \geq$ 1-m. Such forces shall be calculated based on 50-year or 100-year return period of flood or surge. The corresponding wind velocities shall be 260 km/h or 289 km/h respectively.

4. 4RESULTS FROM NUMERICAL ANALYSIS

4.1 Structural Models

The structural response of protective vegetation is simulated using two tree models (of diameters 10″ and 20″ for heights 20′ and 50′ respectively, shown in Fig. 4), subjecting them to hydrodynamic wave loads from two cyclones.

4.2 Material Properties

Table 4: Mechanical Properties of Materials used

Materials	Properties	Values
Timber	Crushing strength	6 ksi
	Modulus of Elasticity	2500 ksi
	Ultimate Strain	0.008

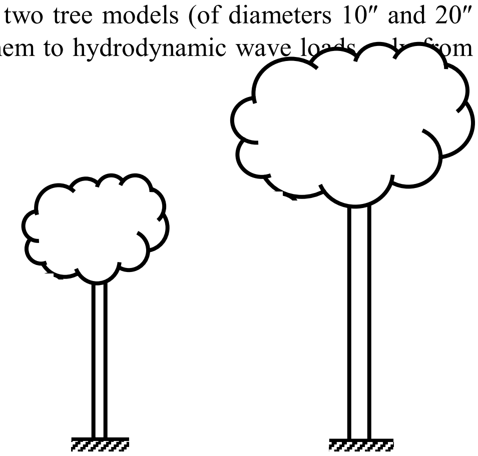

Fig. 4: Elevation of the 20-ft and 50-ft trees

4.3 Loading Condition

Different types of wave load are used for the structural analyses. For protective vegetations, storm surges (i.e., water depth *d*) of 10-ft and 25-ft are used (with significant wave heights of 3-ft and 6-ft and corresponding current velocities) for 10-in diameter 25-ft tree as well as 20-in diameter 50-ft tree.

4.4 Structural Analysis of Protective Vegetation

As mentioned, two types of tree are used to model the protective vegetation in the coastal areas. One of these is 25-ft high with a 10-in diameter, while the other is 50-ft high with a diameter of 20-in. The material properties of both are however assumed to be identical.

4.4.1 Moment Curvature Relationship

The moment vs. curvature relationships for the two trees are derived numerically and shown in Fig. 5. The nature of the two curves is identical, both rising nonlinearly to a peak moment and decaying thereupon without showing much plastic deformation or yielding. This is expected from cross-sections made of timber, whose stress-strain diagram is nonlinear and shows no defined yielding or plastic deformation.

The ultimate moment capacity of the larger section is about eight times the capacity of the smaller section, whose diameter is half the diameter of the larger section. However the corresponding curvature of the smaller section is twice the value for the larger section, both of which correspond to the identical ultimate strain of 0.008

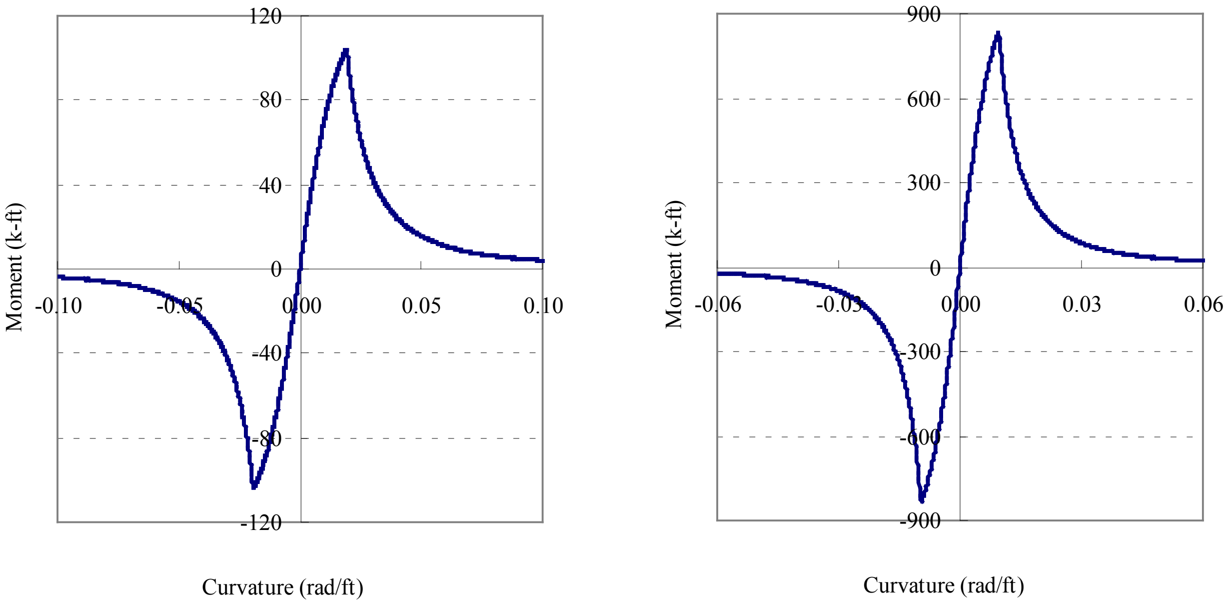

Fig. 5: Moment vs. Curvature for (a) 10-in dia Tree, (b) 20-in dia Tree

4.4.2 Deflections from Wave and Current Loading

Figs. 6and 7 show the variation of the tip deflection of the cantilever trees subjected to storm surges of 10-ft and 25-ft with respective wave heights of 3- and 6-ft respectively, and corresponding current velocities.

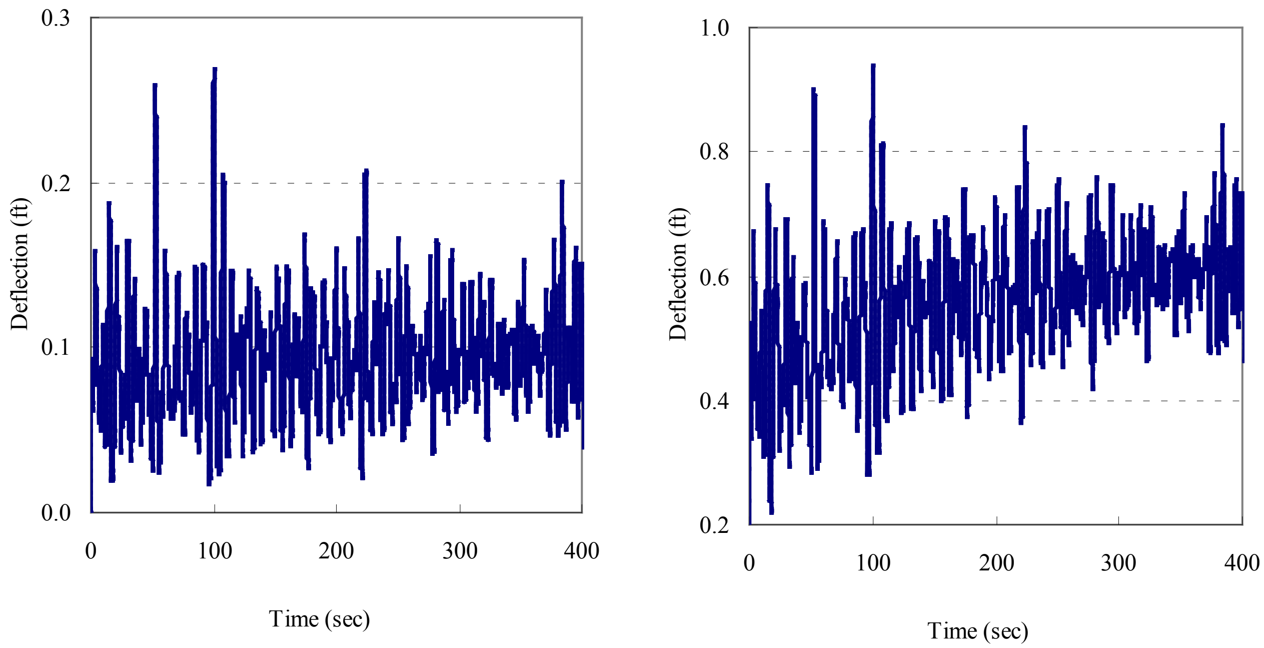

Fig. 6: Tip Deflection vs. Time for 25-ft Tree for
(a) Storm Surge = 10 ft, Wave Height = 3 ft, (b) Storm Surge = 25 ft, Wave Height = 6 ft

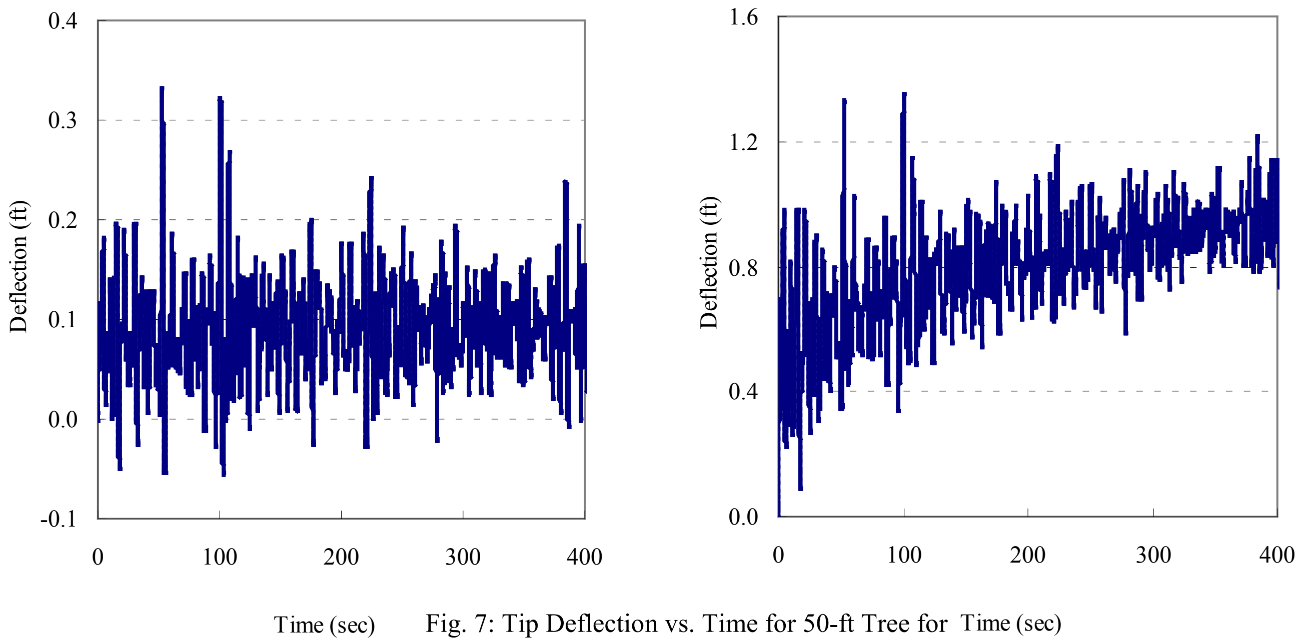

Fig. 7: Tip Deflection vs. Time for 50-ft Tree for
(a) Storm Surge = 10 ft, Wave Height = 3 ft, (b) Storm Surge = 25 ft, Wave Height = 6 ft

4.4.3 Curvature from Wave and Current Loading

Figs. 8 and 9 show the corresponding variations of maximum curvature (at the base). One significant observation here is that in no case does the maximum curvature exceed the curvature corresponding to the ultimate moment for any section.

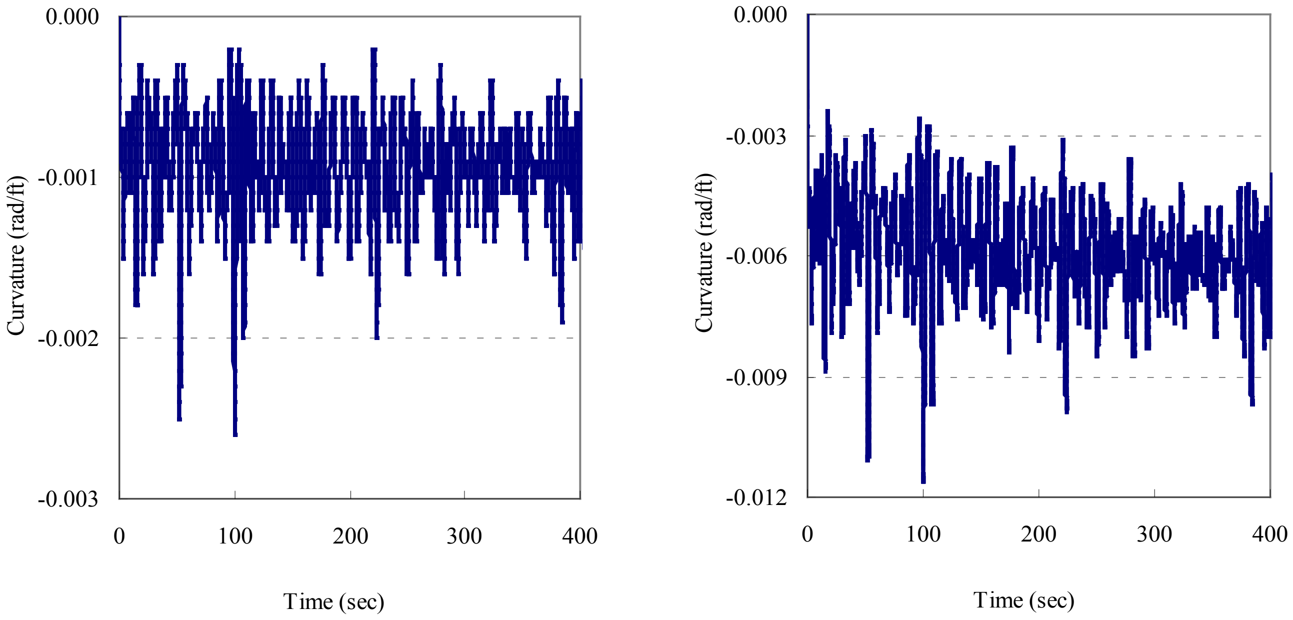

Fig. 8: Maximum Curvature vs. Time for 25-ft Tree for
(a) Storm Surge = 10 ft, Wave Height = 3 ft, (b) Storm Surge = 25 ft, Wave Height = 6 ft

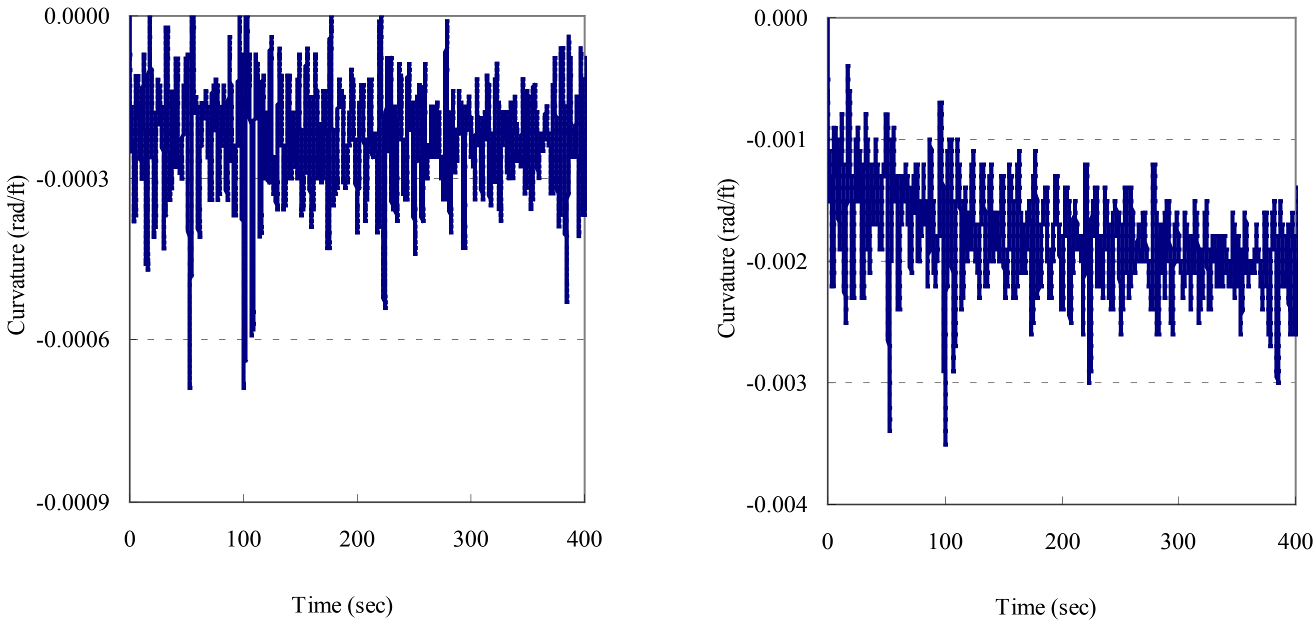

Fig. 9: Maximum Curvature vs. Time for 50-ft Tree for
(a) Storm Surge = 10 ft, Wave Height = 3 ft, (b) Storm Surge = 25 ft, Wave Height = 6 ft

5 CONCLUSIONS

For a 20 in diameter tree moment capacity is much higher than (almost eight times) for the 10 in diameter tree. However, the curvature ductility of the smaller section is larger, due to the identical ultimate strains of the sections. Deflection is highest at the top section of trees as expected, while the maximum curvature is at the bottom segment. For both the 25-ft tree and the 50-ft trees, deflection as well as maximum curvature changes quite significantly with the change of wave height. The deflections and maximum curvatures of the protective structures are found to be well within the limits permitted by the moment-curvature relationships.

Further studies can be done in this area considering actual cyclone data to simulate more real picture. Structural models can also be tested in the laboratory under wave loading to get the experimental result. More detailed analysis on protective vegetation; including their behavior under wave and wind loading, as well as the decay of wave height and current velocity with the density of vegetation can be done. Suitability of different type and size of trees for the best protective vegetation could be found out also by experimenting different types of trees.

REFERENCES

Government of Bangladesh (GoB) (2008). “Cyclone Sidr in Bangladesh: Damage, Loss, and Needs Assessment for Disaster Recovery and Reconstruction.” aReport Prepared by the Government of Bangladesh Assisted by the International Development Community with Financial Support from the European Commission.

Hossain, M. Z., Islam, M. T., Sakai, T, and Ishida, M. (2008).“Impact of Tropical Cyclones on Rural Infrastructures in Bangladesh”. Agricultural Engineering International: the CIGR Ejournal.2

Mazda, Y., Wolanski, E., King, B., Sase, A., Ohtsuka, D. and Magi. M., 1997, “Drag force due to vegetation in mangrove swamps”, Mangroves and Salt Marshes, Vol. 1, pp. 193–199.

Navarre Beach Fishing Pier Design (2006),”Wave Load Calculations, Methodology & Results”

Islam, M. R., 2004, “Living in the Coast: Problems, Opportunities and Challenges”, Working Paper WP011, Dhaka. 2004, Programme Development Office (PDO) and Integrated Coastal Zone Management Plan (ICZMp), pp 13-15

Karim, N. (1999). “Options for Cyclone Protection: Bangladesh Context” Department of Sociology, University of Dhaka.

Khan, S.,R. (2006), “Cyclone Hazard In Bangladesh”

Rahman, M. M., and Biswas, S. K. (2009).“Feasible Solution of Protection and Adaptation Strategy for Coastal Zone of Bangladesh.” Pakistan Journal of Meteorology, 8,9-19.

Saleken, M. (2011).“Response of Low Cost Structure to Wind Load.” B.Sc. Engrg. Thesis, Dept of Civil Engrg., UAP.

Tanaka, N. (2010). “Effectiveness and Limitations of Vegetation Bioshield in Coast for Tsunami Disaster Mitigation” The Tsunami Threat-Research and Technology.161-168.

AN APPROACH FOR SURFACE WATER QUALITY IMPROVEMENT OF RIVERS AROUND DHAKA CITY FOR IRRIGATION USE

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ABSTRACT

Unlike any other city in the world, Dhaka is blessed by a river system around its main city. This river system includes Buriganga, Turag, Balu and Shitalakhya with some adjoining khals. Due to lack of treatment of industrial wastes and inadequate management of municipal waste, the circular water course surrounding Dhaka city is facing serious contaminated phase. Any use of river water becomes unimaginable to city dwellers. It can be apprehended that peripheral river water can be utilised for irrigation purpose like gardening, cultivation of vegetables etc., if they are passed through a process of sedimentation and filtration. The present study deals mainly with the assessment of the present water quality of the peripheral rivers and their beneficial use. Water quality data were collected from DOE and DWASA that have been analyzed and experiment has been conducted at Hydraulic Laboratory of Water Resources Engineering Department, BUET. Based on experiments, a procedure has been suggested to improve the quality of water especially for small scale urban irrigation use. It is hoped that this study will provide a preliminary step towards suggestion of a simplified treatment plant for improving raw water quality of urban rivers to some extent.

Keywords: *River pollution, Peripheral rivers, Filtration, Laboratory analysis, Total solids, Irrigable water*

1. INTRODUCTION

Dhaka is the 9th largest city in the world has a population of more than 15 million and growing at an annual rate of around 5% which is one of the highest amongst Asian cities. The city is situated at the center of the country. The river network around Dhaka city includes Buriganga, Turag, Shitalakhya and Balu and some khals which are linked to the river system such as Tongi khal, Begunbari khal, Dholai khal, kollanprur khal etc. Due to inadequate management of solid waste producing from industry, household and other sources the circular water body surrounding Dhaka city is highly polluted. The peripheral rivers have become a disgusting part for the city dwellers. River pollution occurs mainly due to discharge of industrial effluents and municipal sewage. One of the important sources is textile industrial effluents. A World Bank study said four major rivers near Dhaka -- Buriganga, Shitalakhya, Turag and Balu -- receive 1.5 million cubic metres of waste water every day from 7,000 industrial units in surrounding areas and another 0.5 million cubic metres from other sources. At present condition the quality of water is so worsened that they are called “Biologically Dead” during dry season which creates hazardous impacts on ecological system and on the environment around. More than 60,000 cubic metres of toxic waste from textile dying, printing, washing and pharmaceuticals enter rivers every day. The level of water quality parameters such as Dissolved Oxygen, Biochemical Oxygen Demand, turbidity, Chemical Oxygen Demand, pH, Ammonia, Chromium, coliform etc. concentration is too high to use for irrigation.

Water supply problem is acute in this city because of the failure of combined use of surface water and ground water. The treated surface water contains only 15% total supply of DWASA. For the remaining 85% we have to depend on ground water supply. As a result groundwater level is lowering day by day. The condition is so worse because the surface and subsurface geologic formation under Dhaka city are not favorable for replenishment and storage of ground-water against heavy withdrawal. Groundwater level in 2007 was about 60 m below ground level, which were only a few meters below ground level during the 1970s. At present groundwater level is declining at even greater (>2 m/year) rate. DWASA cannot fulfill the demand. While the city needs 2.2 billion liters of water a day, it can produce 1.9-2 billion. So it is necessary to make a combined use of groundwater and surface water. This study will help to provide a possible solution for the use of surface water based on laboratory experimental test and will also help to take necessary steps for improving raw water quality.

2. METHODOLOGY

For the study following tasks have been done:

2.1 Data Collection

Water quality parameter data of previous years has been collected from Department of Environment (DOE) and laboratory of Dhaka Water Supply & Sewage Authority (DWASA). Some water quality parameters such as Dissolved Oxygen, Biochemical Oxygen Demand, pH, Nitrogen, Fecal Coliform have been analysed by graphical representation and compared with the recommended surface water quality for irrigation purposes according to Bangladesh Environment Conservation Rules 1997. These water Quality parameters were selected because of their significance on irrigation purpose.

2.2 Laboratory Experiment

Water has been collected from some random points of Buriganga, Balu, Turag and Shitalakhya during both the dry season (January) and monsoon (June) season. Table 1 shows the location of sample collection.

Table 1: Location of sample collection during monsoon and dry period

River Name	Location
Buriganga	Lohar Bridge
Balu	Ichapura Bridge
Turag	Tongi Bridge
Shitalakhya	Rupganj at Narayanganj

Experiment has been carried out through the process of sedimentation and filtration. For the setup of a filter some representative soil samples are collected from the place in front of the Nazrul Islam Hall, BUET which are used as filtering soil for the tube-well that was dug into that place at 2012. The 1st layer of soil is collected from at a depth of 400-450ft, the 2nd layer of soil is collected from at a depth of 330-340ft, and the 3rd layer of soil is collected from at a depth of 310-330ft. The Fineness Modulus of the first, second and third layers are 1.44, 1.72 and 2.25 respectively.

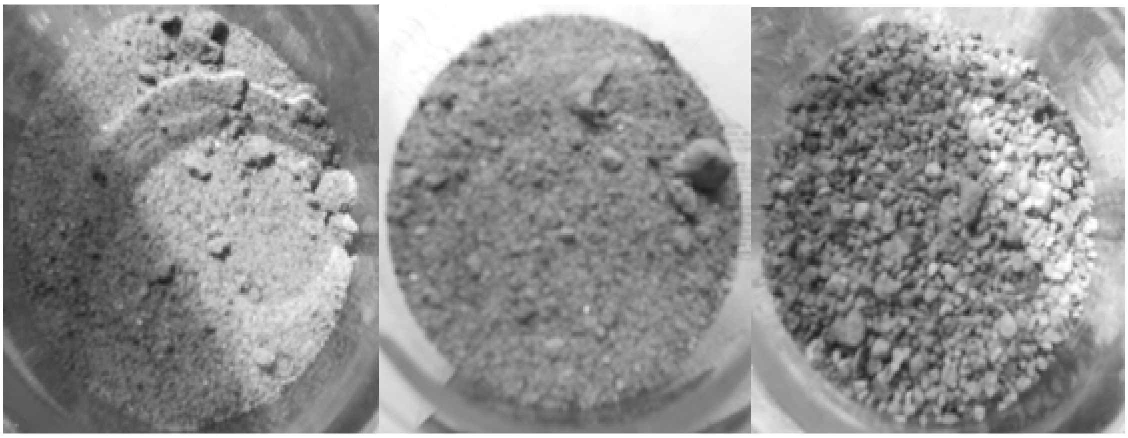

Figure 8: Filter material sample used for 1st, 2nd and third layer of filter

2.2.1 Laboratory scale Filtrometer

For the setup of a filter equipment named "Filtrometer" at Hydraulic Laboratory of Water Resources Engineering Department in BUET is used. Filter materials are placed into the Filtrometer by one layer to another. The finer materials are placed at the bottom and coarser materials are placed at the top. The depth of each layer is 10 cm. The setup is shown in Figure 2.

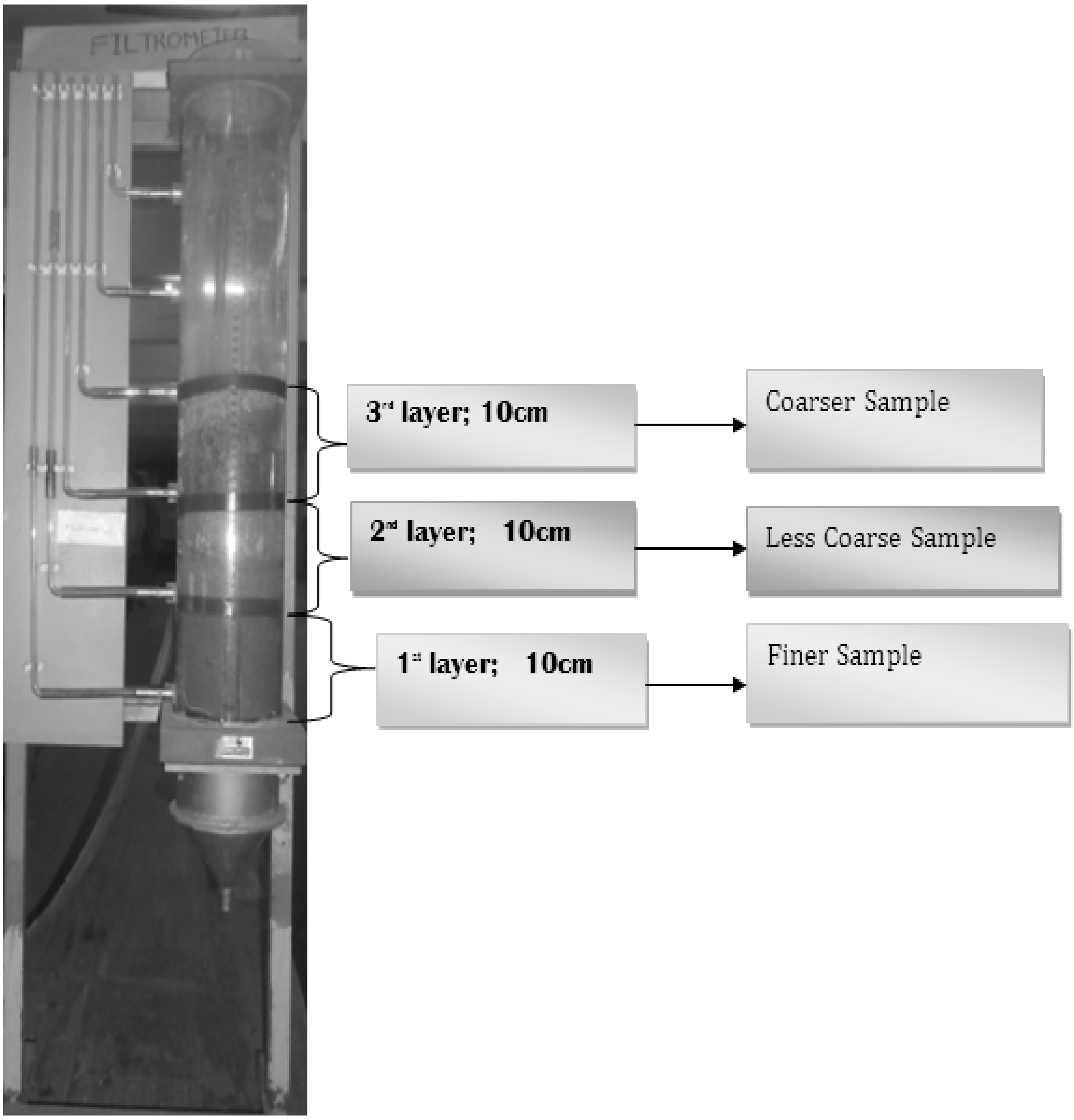

Figure 9: Three layers of sand in Filtrometer

2.2.2 Filtration Process

After placing the soil sample into the filter, normal tap water is poured into the soil to wet. After 24 hours, filtration process is started. The water collected at various locations is poured into filter for filtration. Prior to filtration each samles was placed for a while for sedimentation.

2.2.3 Laboratory analysis for total solids

For all the samples before and after filtration, Total Solids have been measured. For this analysis each of 100 ml are put into separate beakers. They are kept in oven for 24 hrs and at a temperature of 103°C. Before that, weights of all beakers are measured. After 24 hrs the weight of the beakers are measured again. The differences between these two give TS.

Figure 3: Filtration process and oven-dry process

3. WATER QUALITY OF THE PERIPHERAL RIVERS

The chemical, physical and biological characteristics of water can be referred as the water quality and water quality assessment means the overall process of evaluation of these three characteristics of the water. The water quality in a river system is closely related with its hydraulic condition and pollution loads.

3.1 Water Quality Parameters

The water quality of rivers surrounding Dhaka City has been deteriorated to an extreme extent. Factories, industries, tanneries established around the rivers play the major roles for such pollution. In addition, due to densely populated Dhaka city, domestic and municipal wastes have contributed much to make the situation more hazardous.

3.1.1 Dissolved Oxygen

Dissolve Oxygen analysis measures the amount of gaseous oxygen (O_2) dissolved in an aqueous solution. Oxygen gets into water by diffusion from the surrounding air, by aeration (rapid movement), and as a waste product of photosynthesis. Natural stream purification processes require adequate oxygen levels in order to provide for aerobic life forms. As dissolved oxygen levels in water drop below 5.0 mg/l, aquatic life is put under stress.Oxygen levels that remain below 1-2 mg/l for a few hours can result in large fish kills. There is no maximum amount or upper limit for dissolved oxygen.

According to Bangladesh Environment Conservation Rules 1997, the standard limit of DO in case of water usable for irrigation is 5 mg/l or more. Dissolve Oxygen measured by DOE during March and April 2010, 2011 along peripheral rivers around Dhaka city is shown in Figure 4.

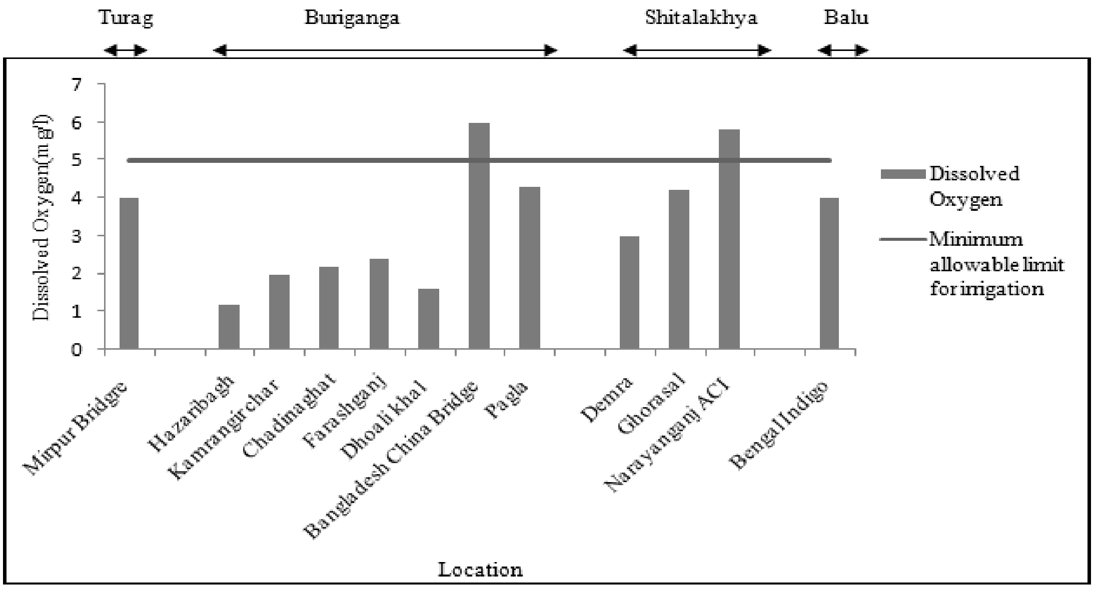

Figure 4: Maximum DO in the rivers at various locations during March and April 2010, 2011

3.1.2 Biochemical Oxygen Demand

The amount of oxygen required by microorganisms to oxidize organic wastes aerobically is called Biochemical Oxygen demand (BOD). When biodegradable organic matter is released into a water body, microorganisms feed on the wastes, breaking it down to organic and inorganic substances. When this decomposition takes place in an aerobic environment, it produces non-objectionable, stable end products and in the process draws down the dissolved oxygen content of water.

When insufficient oxygen is available or when oxygen is exhausted by the decomposition of wastes, different set of microorganisms carry out the decomposition anaerobically producing highly objectionable products. According to Bangladesh Environment Conservation Rules 1997, the standard limit of BOD in case of water usable for irrigation is 10 mg/l or less. BOD_5 measured by DOE during March and April 2010, 2011 along peripheral rivers is shown in Figure 5.

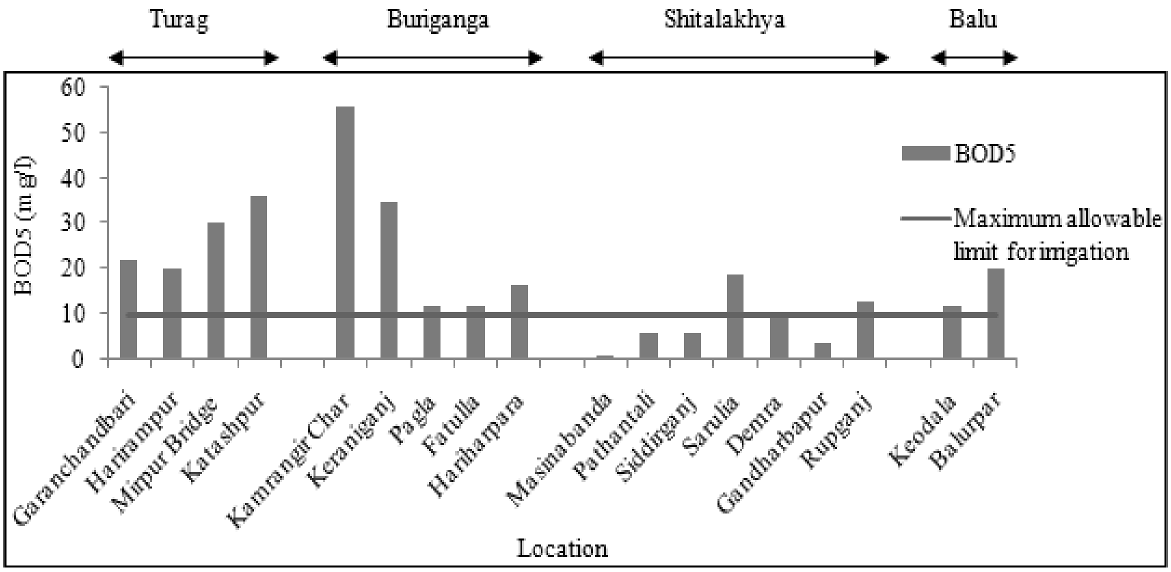

Figure 5: Maximum BOD_5 in the rivers at various locations during March and April 2010, 2011

3.1.3 Total Solids and Total Dissolved Solids

Total solids refer to the matter that remains as residue upon evaporation and drying at 103 to 105°C. Total Solids include Total Suspended Solids and Total Dissolved Solids. Levels of total solids that are too high or too low can reduce the efficiency of wastewater treatment plants, as well as the operation of industrial processes that use raw water.

Total dissolved Solids (TDS) comprise inorganic salts and small amount of organic matter that passes through the filter. In potable water most of the solids remain in dissolved form and ranges from 20 to 1000 mg/L. To determine whether water is suitable for domestic purpose, it is required to know how much solid it contains. Water with high dissolved solids generally is of inferior palatability and may induce unfavorable physiological reaction in the user. From various investigations it is seen that Buriganga, Balu, Turag and Shitalakhya contain very high Total Solids during dry season because of low flow of water and lack of rainfall.

3.1.4 pH

pH value denotes the acidic or alkaline condition of water. pH is usually represented by a scale ranging from zero to 14, with 7 being neutral. A controlled value of pH is desired in water supplies, sewage treatment and chemical process plants. In water supply pH is important for coagulation, disinfection, water softening and corrosion control. In biological treatment of wastewater, pH is an important parameter, since organisms involved in treatment plants are operative within a certain pH range.

According to Bangladesh Environment Conservative Rules (1997), an acceptable range of pH in irrigation water is 6.5-8.5. pH measured by DOE during March – April 2010, 2011 along peripheral rivers is shown in Figure 6.

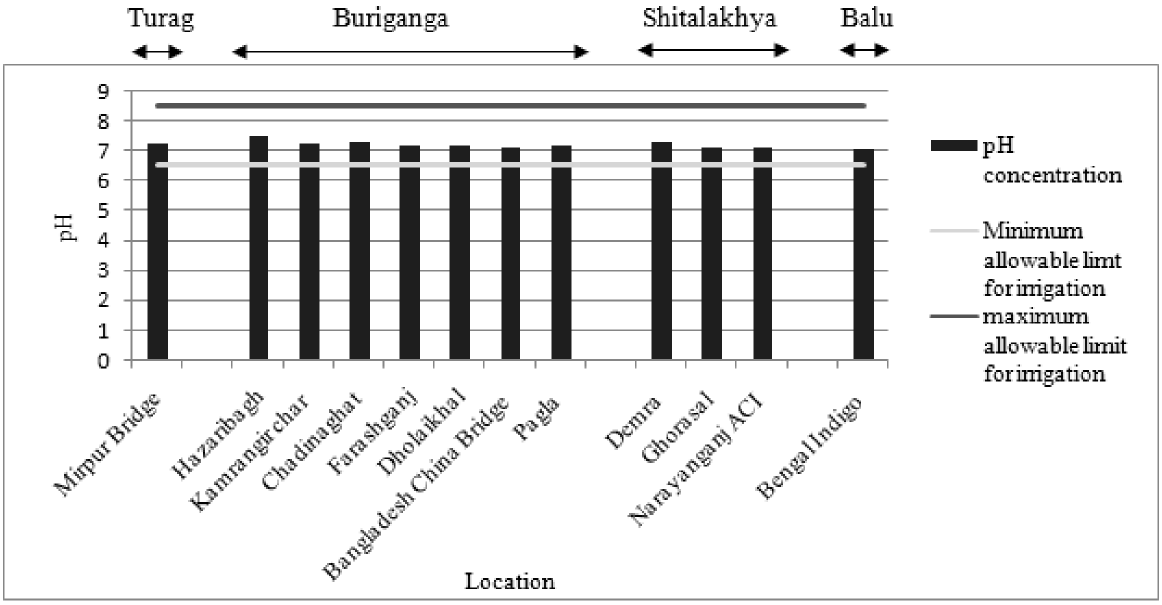

Figure 6: Profile of maximum pH concentration in the rivers during March and April 2010, 2011

3.1.5 Nitrogen

Depending on water properties, various inorganic nitrogen compounds may be found in water. In aerobic waters nitrogen is mainly present as N_2 and NO_3-, and depending on environmental conditions it may also occur as N_2O, NH_3, NH_4+, HNO_2, NO_2- or HNO_3. Ammonium, nitrate and nitrite play the most important role in biochemical processes, but some organic nitrogen compounds in water may also be of significance. Total nitrogen represents the sum of organic and inorganic nitrogen compounds.

Nitrogen-containing compounds act as nutrients in streams and rivers. Nitrate reactions [NO_3-] in fresh water can cause oxygen depletion. Thus, aquatic organisms depending on the supply of oxygen in the stream will die. The major routes of entry of nitrogen into bodies of water are municipal and industrial wastewater, septic tanks, feed lot discharges, animal wastes (including birds and fish) and discharges from car exhausts.
According to Bangladesh Environment Conservation Rules 1997, the standard limit of Ammonia as Nitrogen in case of water usable for pisiculture is 1.2 mg/l. NH_3-N measured by DOE during March – April 2010, 2011 along peripheral River System around Dhaka City is shown in figure 7.

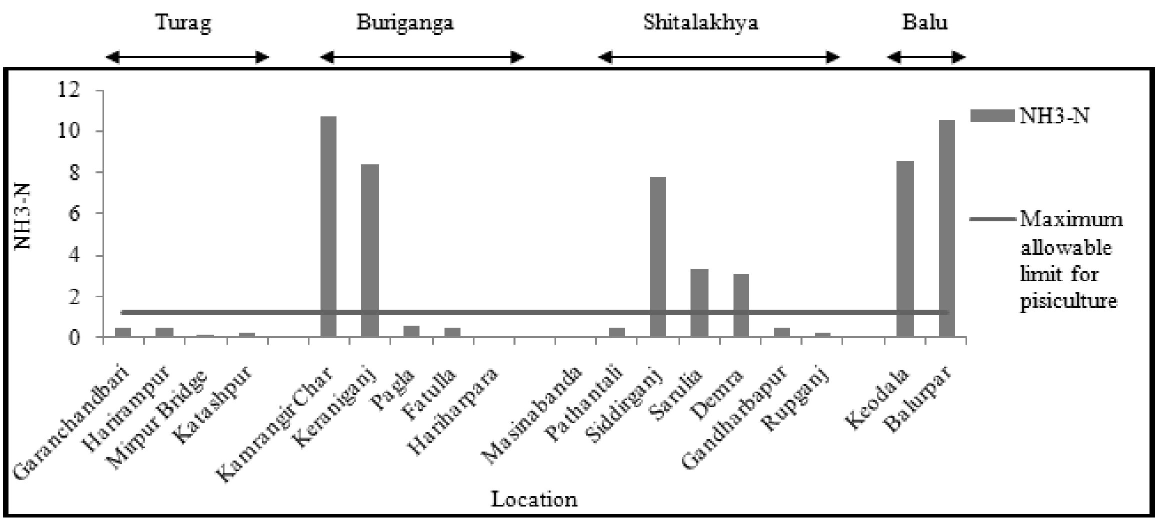

Figure 7: Maximum NH3-N concentration in the rivers during March and April 2010, 2011

3.1.6 Bacteriological Characteristics

Bacterial examination of water indicates the degree of pollution. Water polluted by sewage contains one or more species of disease producing pathogenic bacteria. Pathogenic organisms cause water borne diseases, and many non pathogenic bacteria such as E.Coli, a member of coliform group, also live in the intestinal tract of human beings.

Long term observation of coliform is not available. It was measured by DOE at several locations of the rivers during Feb'04. High concentration of fecal coliform can be expected in the entire river reaches since they are receiving huge domestic sewage from the city & its outsides.

According to Bangladesh Environment Conservation Rules 1997, the standard limit of Total Coliform in case of water usable for irrigation is 1000 per 100 ml of water or less. Fecal coliform measured by DOE during February'04 along peripheral River System around Dhaka City is shown in figure 8.

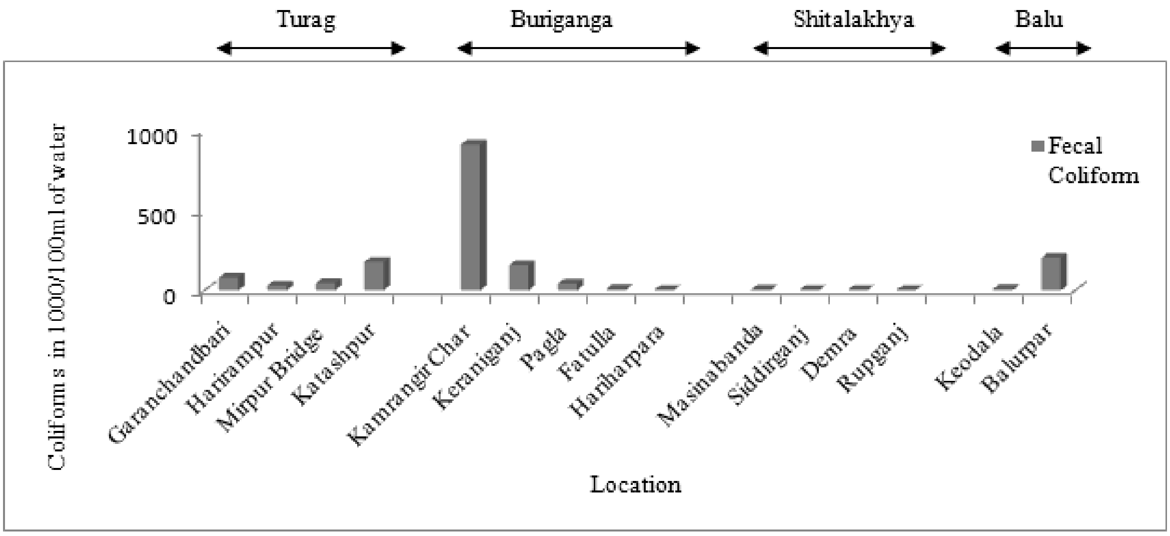

Figure 8: Fecal Coliform in the river at various locations during February in 2004

3.2 Laboratory Experiment

3.2.1 Observation on appearance (Color)

Collected raw water samples show different type of colors .Some of them are yellowish, some are blackish, and some are black. But after present treatment (sedimentation and filtration), the color has been changed to a clear appearance. The comparison of color between before and after the treatment is shown in Table 2.

Table 2: Color Analysis before and after Filtration

River	Collection Date	Raw Sample	After Filtration
Buriganga	31-6-2012	Blackish	Satisfactory result
Balu	24-6-2012	yellowish	Satisfactory result
Turag	24-6-2012	yellowish	Satisfactory result
Shitalakhya	31-6-2012	yellowish	Satisfactory result
Buriganga	23-1-2013	Black	Satisfactory result
Balu	25-1-2013	Black	Satisfactory result
Turag	24-1-2013	Black	Satisfactory result
Shitalakhya	24-1-2013	Black	Satisfactory result

The change of color of the samples before and after filtration is shown in the Figures 9, 10, 11 and 12.

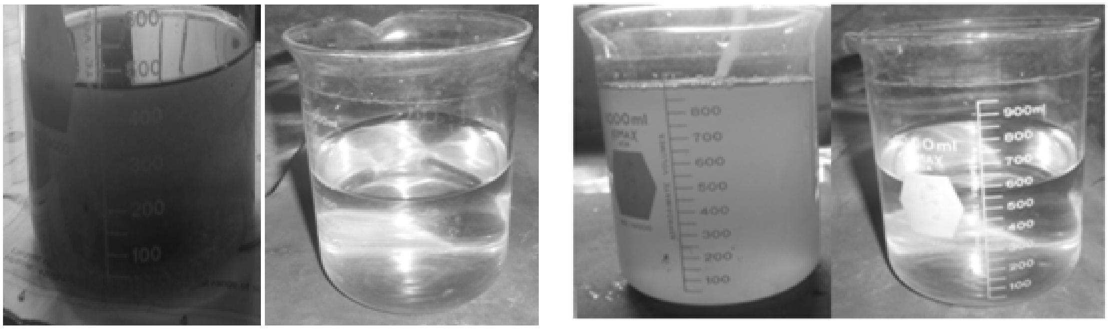

Figure 9: Before and after filtration of the sample of Buriganga (dry season and monsoon season)

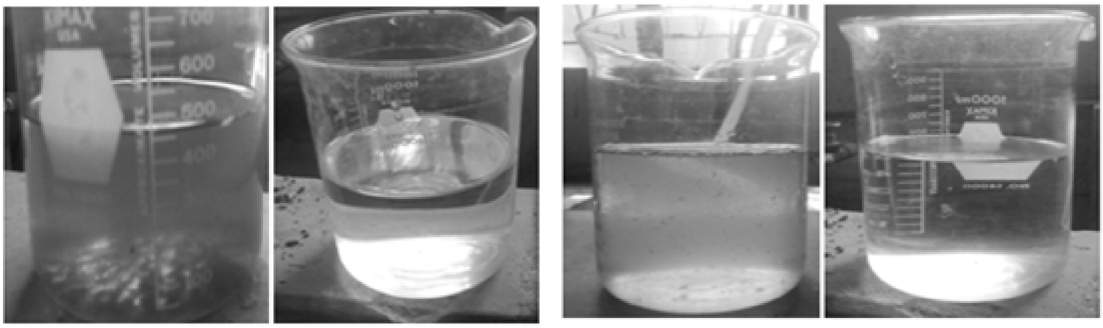

Figure 10: Before and after filtration of the sample of Shitalakhya (dry season and monsoon season)

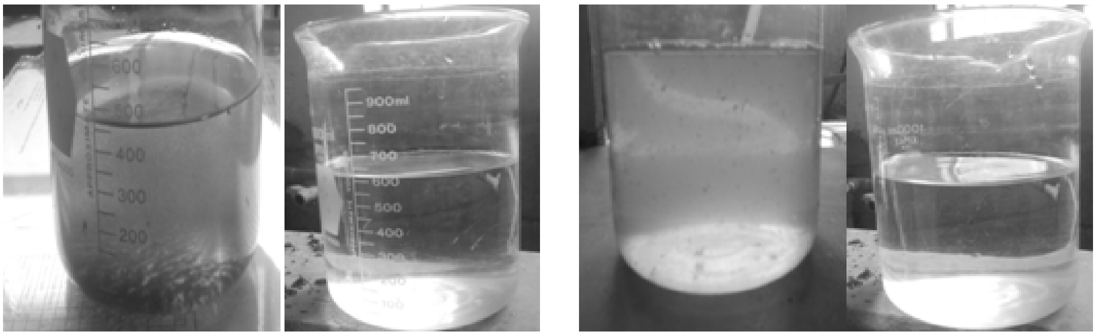

Figure 11: Before and after filtration of the sample of Turag (dry season and monsoon season)

Figure 12: Before and after filtration of the sample of Balu (dry season and monsoon season)

3.2.2 Odor Analysis

The samples that are collected during dry period had acute odor and that collected from monsoon period had slightly odor. The comparison of odor between before and after filtration is listed Table 3.

Table 3: Odor Test Analysis

Sample From	Collection Date	Odor Before Filtration	Odor After Filtration
Buriganga	31-6-2012	Slightly Odor	Satisfactory result
Balu	24-6-2012	Slightly Odor	Satisfactory result
Turag	24-6-2012	Slightly Odor	Satisfactory result
Shitalakhya	31-6-2012	Slightly Odor	Satisfactory result
Buriganga	23-1-2013	Acute Odor	Satisfactory result
Balu	25-1-2013	Acute Odor	Satisfactory result
Turag	24-1-2013	Acute Odor	Satisfactory result
Shitalakhya	24-1-2013	Acute Odor	Satisfactory result

3.2.3 Total Solids Analysis

Analysis carried out for the collected water samples. A relative comparison of TS between before and after filtration both in dry season and monsoon season are shown in Figure 13 and 14 respectively. Storm water dilution effect is prominent as apparent from these results.

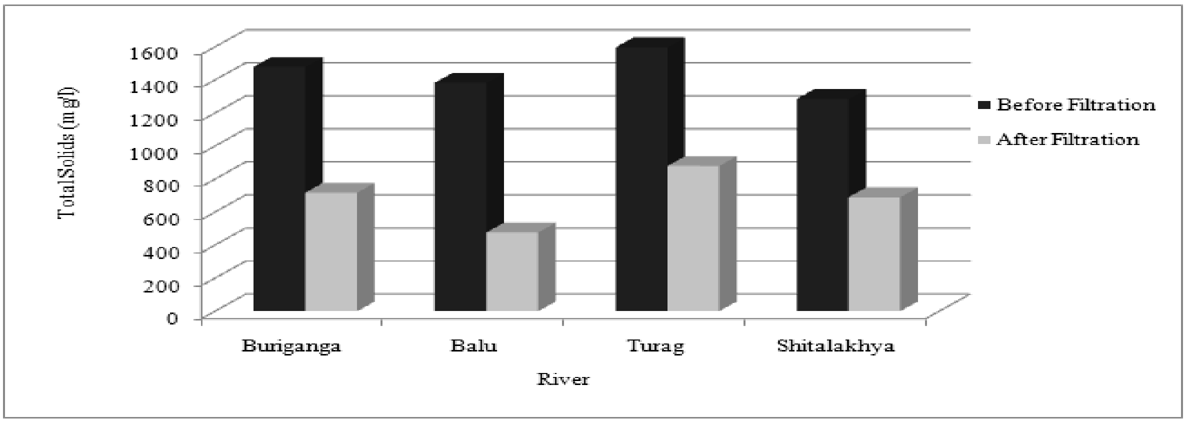

Figure 13: Comparison of Total Solids (TS) between before and after filtration (dry season)

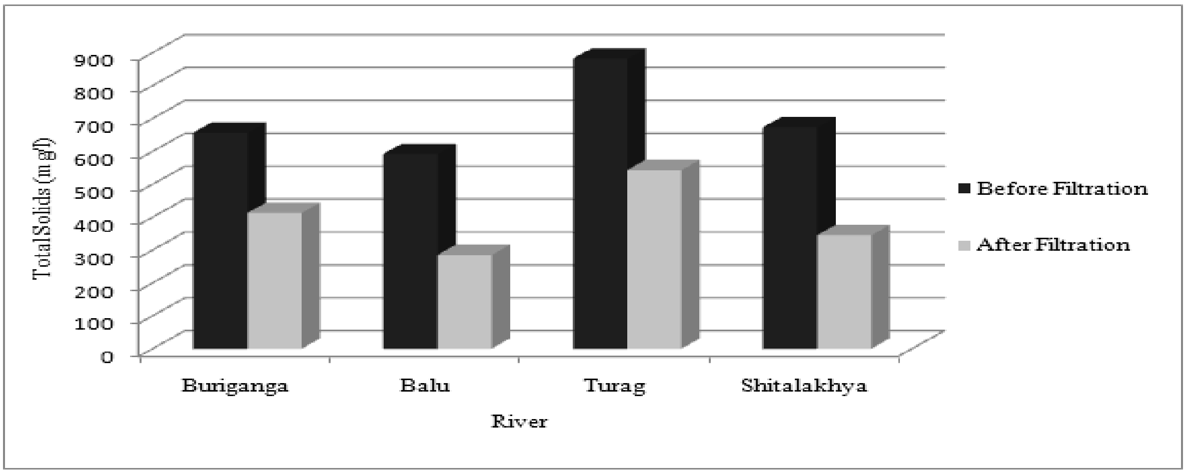

Figure 14: Comparison of Total Solids (TS) between before and after filtration (monsoon season)

4. CONCLUSIONS

Analysis of collected water quality parameter data, comparison with the standard limit that usable for irrigation according to "Bangladesh Environment Conservation Rules 1997, Schedule 3(A)" and laboratory experiment show that the quality deteriorates severly during the dry period than monsoon period. Dissolved oxygen goes below 5mg/l in case of most of the locatios during dry season. As such the aquatic life is put under stress and fish cannot grow in this water. pH of the river system is satisfactory because it lies within acceptable limit of irrigation purpose of 6.5-8.5. Biochemical Oxygen Demand (BOD at 20°C) is high except a few locations and exceeds the allowable limit of 10mg/l or less. Ammonia concentration is high in some locations. High concentration of coliform exists in the entire river system. It exceeds the maximum limit of 1000/100ml of water usable for irrigation. So it can be said that river water around Dhaka city is not suitable enough to use in irrigation purposes.

A simplified procedure of improving the surface water quality has been suggested (Figure15). Laboratory scale tests and analysis show satisfactory improvement on the quality of water. According to ECR 1997, Schedule 3(A), standard limit of Electrical conductivity for irrigation water is 2250 µS/cm (at a temperature of 25°C); which means that the standard limit of Total Dissolved Solid for irrigation water should be within 1400 to 1500mg/l or less (as a rough approximation). Laboratory analysis shows that Total Solids after filtration ranges between 250mg/l to 900 mg/l i.e. TDS ranges between the standard limit (Total Solids= Total Dissolved Solids+ Total Suspended Solids). It is suggested that water, are within 250 to 900mg/l, as treated under present procedure can be used for irrigation purpose. Raw water treatment plant is necessary to improve the water quality for urban irrigation use like winter vegetation, gardening etc.

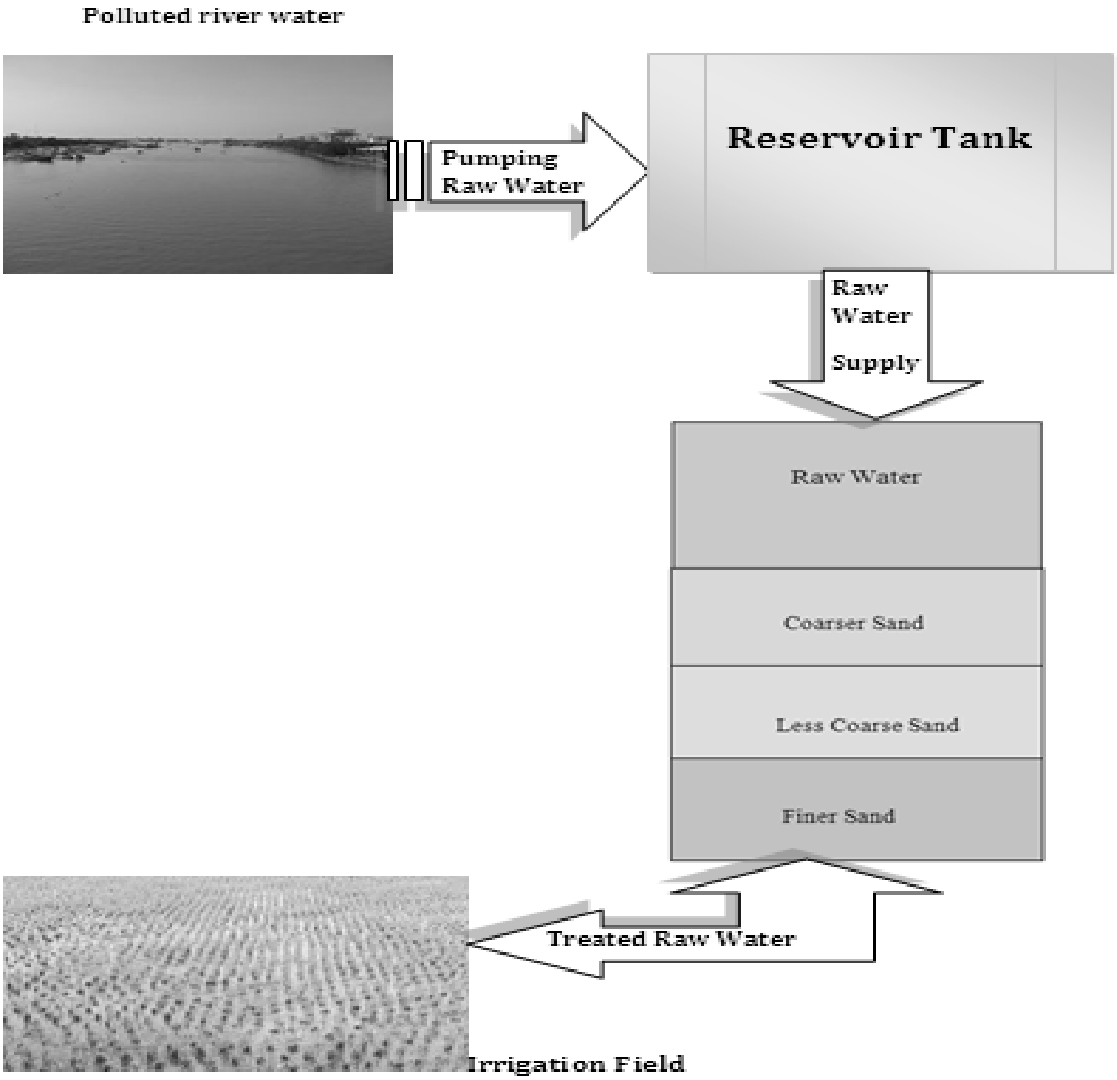

Figure 15: Simplified procedure for the improvement of water quality for irrigation use

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REFERENCES

Begum, D.A. and Ahmmed, K.M.T (2010). "Water Quality Aspects in and around Dhaka City." Proc. of International Conference on Environmental Aspects of Bangladesh". ICEAB10, Japan.

Bahauddin, K.M.and Hossain, N. (2013). "Integrated Water Resource Management for Mega City: A Case Study of Dhaka City, Bangladesh".

Dhaka_Wikipedia, the free encyclopedia, http://en.wikipedia.org/wiki/Dhaka

Kamruzzaman , M. and Zaman, M. M. (2007). "Water Pollution by Different Industries around Dhaka City," B.Sc.Engg. Thesis, Bangladesh University of Engineering and Technology, p. 52-54.

MOEF (Ministry of Environment and Forest), Government of the People's Republic of Bangladesh (1997). "The Environment Conservation Rules, 1997".

Motuza, M.R., Selmi, S., Fatmi, M.R. and Rabbani, G.A. (2011). "Potentiality of Rain Water Harvesting in Dhaka: A Greener Approach." Proceedings of the International Conference on Environmental Technology & Construction Engineering for Sustainable Development. ICETCESD-2011, March 10-12, 2011.

Paul, R(2009). "Water Security in Dhaka City-An initiative of Government of Bangladesh". 6th Governing Council Meeting of Asia Pacific Water Forum, APWF, Orchard Hotel, Singapore.

Rahman, A., Zafor, M.A. and Kar S. (2012). "Analysis and Comparison of Surface Water Quality Parameters in and around Dhaka City." International Journal of Civil Engineering and Technology, IJCIET, Vol. 3(2), 07-15, https://www.academia.edu/2472881/

Sandec-Skat (1996). "Surface Water Treatment by Roughing Filters - A Design, Construction and Operation Manual. " p.180.

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COASTAL HAZARDS AND COMMUNITY-COPING METHODS IN SOUTH-WEST COASTAL REGION OF BANGLADESH

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ABSTRACT

Addressing the most vulnerable coastal communities in Bangladesh, this paper explores peoples' perception and vulnerabilities to coastal hazards. At the same time, it investigates the methods that communities apply to cope with different coastal hazards. This research relied on intensive field investigation where two villages were selected from two for questionnaire survey. Sampling was taken from kalikabari villages in Morrelganj upazilla, Bagerhat and Narayanpur village in Kaliganj upazilla, Satkhira that were the most badly natural hazard affected areas during all previous natural disaster of Bangladesh. Findings revealed that people perceived an increase in both the intensity of hazards and their vulnerabilities. In spite of having a number of socio-economic and location factors enhancing their vulnerabilities, the community is creating their ways to cope with these hazards. It is also important to note that in different aspects of life community coping varies with the variation of hazard. The efforts of development organizations need to ensure adequate accommodation in cyclone shelters, which should be connected by a better transportation system and location of the center should be selected by the community's participation.

Keywords: Coastal hazards, Community-coping methods, Coastal region, vulnerability

1. INTRODUCTION

The coastal area is recognized as a zone subject to intensive human use (Islam, 2008a, b). The coastal areas of Bangladesh have perfect resemblance with such intensive uses. Presently, these areas are being used for agriculture, livestock rearing, fishing, shrimp culture, and salt production. Coastal areas also comprise sites of export processing zones (EPZ), airports, land ports, harbors, and tourism. Unfortunately, however, these areas are highly vulnerable to both natural and man-made hazards and disasters like coastal cyclone, tidal surge, flooding, river bank erosion and drought, etc. In addition to the different hazards in terms of socio-economic condition of households in the coastal area are poor income levels, and poor housing and sanitation conditions have enhanced the vulnerabilities of coastal communities. The vulnerability of the coastal areas of Bangladesh is aggravated by climate change and its impact. Studies by the Intergovernmental Panel on Climate Change (IPCC) have already suggested that being a deltaic plain, climate change related sea-level rise and other hydro-meteorological effects could have a catastrophic impact on the coastal mangrove ecosystems such as Sundarban's and surrounding human settlements (IPCC, 2001). Furthermore, all over the world, along with climate change, the frequency and intensity of hydro-meteorological disasters per year is increasing over the time (UNISDR 2007; IPCC, 2001). Disaster-related records of Bangladesh also prove this; for instance, the number of cyclones tripled over the last 50 years (Islam 2004). There is no doubt that the people living in coastal areas are more vulnerable to these disasters than the people in other areas. The devastating tsunami that occurred in 2004, hurricane Katrina in 2005, cyclone Sidr in 2007 and Nargis in 2008, which killed millions of people in coastal areas, are obvious recent examples of climate extremes and the associated vulnerabilities of coastal residents.

Out of the 64 districts in Bangladesh, 19 are delimited as coastal areas. Figure 1 shows the location of coastal areas of Bangladesh. The population of this area is 36.8 million and more than half of them (52%) are poor and about 41 % is below the age of 15 (Islam 2008a, b). In spite of being a very poor and being vulnerable to different kinds of coastal hazards and disasters, with their age-old indigenous knowledge and perceptions, these people are coping with various hazards and disasters and passing with relentless struggle. However, it is forecasted that climate change and its associated hazards will affect the adaptive capacities of the population, which is already overburdened by absolute poverty, recurring natural disasters, and governance –related issues. Hence, studying the existing community-based coping methods to various coastal hazards and climate-related disasters assumes important for designing the appropriate capacity-building interventions. Furthermore, empirical study addressing local-level coastal communities is rare in Bangladesh.

Therefore, this study intends to provide a brief overview on the coastal hazards in Bangladesh and the current efforts to minimize the adverse impacts on life and livelihood of the coastal population.

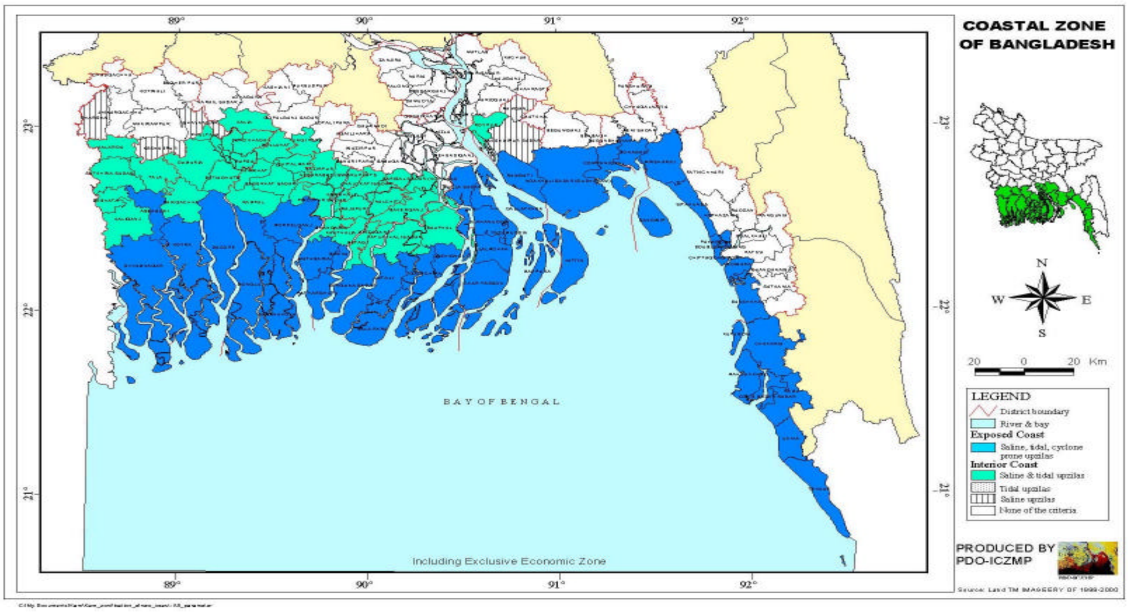


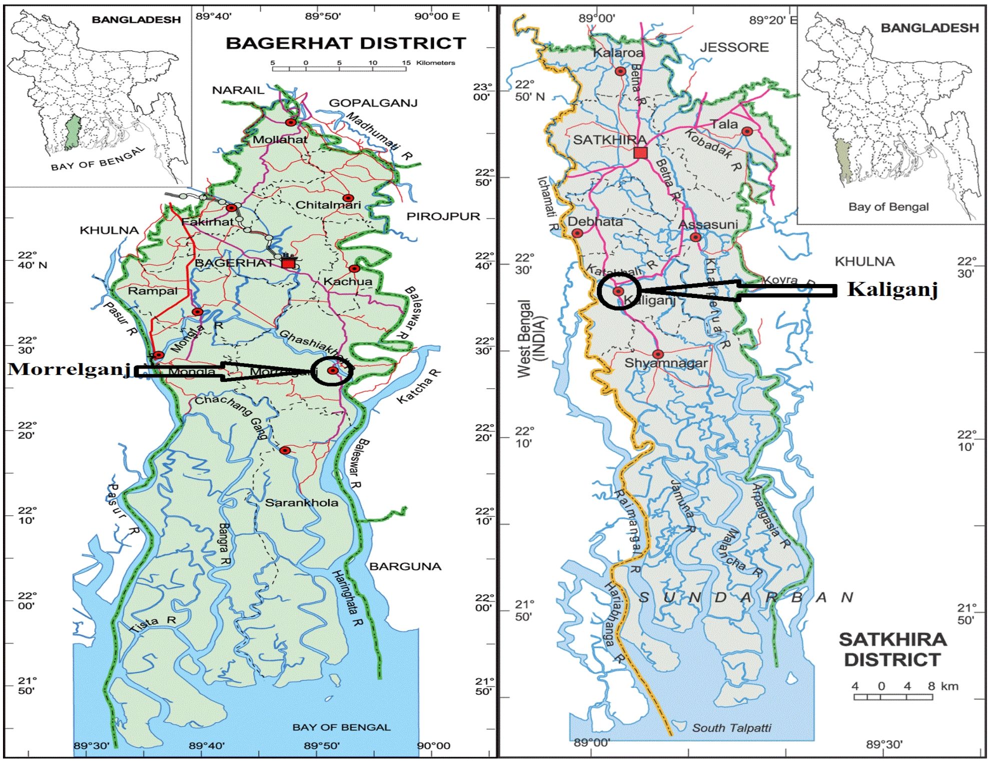
Figure 1: Coastal zone of Bangladesh (MoWR, 2006)

This study finds the most vulnerable livelihood parameters of the local people and their opinion about the natural disaster problem. It also finds the potential effect that might occur to the livelihood of the local people and local coping methods. This study will help to make future decisions with more accuracy and will bring benefits to the local people, as their voice is associated about the solving of the problem that occurs due to natural hazards. It may also help the policy makers to take proper initiatives to solve the problem of water supply, sanitation, health, shelter, food and all other problems that occur due to various natural hazards. It is expected that this would be a realistic aid to the intervention of development organization to manage coastal hazards and to enhance the community's preparation and response ability. The objectives of the study are to examine the coastal community's perception to the hazards and their local coping method against the effect of various coastal hazards.

2. METHODOLOGY

2.1 Study Area

This research relied on intensive field investigation and questionnaire survey of the local people, where sampling was taken from two villages which are natural hazard vulnerable areas of Bangladesh. Two selected study areas are Kalikabari village in Morrelganj upazilla, Bagerhat and Narayanpur village in Kaliganj upazilla, Satkhira (Figure 2). Morrelganj Upazila is situated in the south-eastern part of Bagerhat district with an area of 460.91 sq km, is bounded by BagerhatSadar and Kachua upazilas on the north, sarankhola and mathbaria upazilas on the south, Pirojpur Sadar and Bhandaria upazilas on the east, Rampal and Mongla upazilas on the west. Main rivers: Baleshwar, Ghasiakhali, Panguchi and Bhola. It lies in 22°27′N and 89°51.5′E in Bangladesh. On the other hand, Kaliganj is situated in south-western part of Satkhira district with an area of 333.79 km^2 (128.88 sq mi), is bounded by Debhata and Assasuni upazilas on the north, Shyamnagar upazila on the south, Assasuni and Shyamnagar upazilas on the east, West Bengal of India on the west. Main rivers are Ichamati, Kakshiali, Kalindi and Little Jamuna. Kaliganj is located at 22.4500°N 89.0417°E in Bangladesh.



(a) Location of Morrelganj in Bagerhat (b) Location of Kaliganj in Satkhira

Figure 2: Location of the study areas

2.2 Methodology of the Study

To analyze the impact on livelihood parameters and coping methods, a brief overview was assembled of the coastal hazard and the current efforts to minimize the adverse impacts of the hazards. This study has conducted for two villages contain about 2182 number of people of which about 10% of populations were surveyed.

Table 1 Parameters and its subcomponent regarding questionnaire survey

Parameters	Sub-components/ Survey considerations
Socio-economic condition	Age group, gender distribution, education, housing condition, toilet condition, occupation, income level per month
Hazard type	Cyclone, tidal surge, flood, riverbank erosion, drought, etc.
Shelter condition	Existing housing pattern, impact of hazards, coping methods.
Water supply system	Sources of water, usability of water, impact of hazards, coping methods, etc
Sanitation system	Existing toilet condition, impact of hazards, coping methods
Health condition	Type of diseases during hazard, treatment methods, stocking medicine, etc.
Food management	Availability of food, food storage during hazard, coping methods

From the questionnaire survey, people were questioned about their various livelihood parameters. They were questioned about their socio-economic condition such as number of family member, their age group, gender distribution, educational background, housing condition, toilet condition, occupation and their income level per month. People were also questioned about the change of intensity and their livelihood vulnerability, percentage of occurrence of various natural disasters and their coping methods. They were questioned about the impact of natural hazards on shelter, water supply system, sanitation system, health condition and food. The base line condition of each parameter is surveyed. During hazards, their local coping methods against the mentioned livelihood parameters were questioned as a part of the survey. Besides the structural questionnaire survey, semi-structural interview and discussions with key informants were conducted. Furthermore, focused group discussions were organized in different villages where people from different professions and strata attended and shared their experiences and sufferings during natural disaster. Along with the questionnaire survey these discussion provided wide views and opinions of people at grass root level and enriched the data bank of this research. Table 1 demonstrates the parameters and its sub-components regarding questionnaire survey.

3. RESULTS AND DISCUSSIONS

Results were prepared based on the survey conducted at Morrelganj in Bagerhat and at Kaliganj in Satkhira which are cyclone affected areas in Bangladesh. The people in these areas are bound to face cyclone almost every year. Since the shelter, food, water supply, sanitation and health issues are the most sensitive and crucial during cyclone, this study concentrates on these issues only.

3.1 Case Studies

Morrelgonj and Kaliganj both are located in the south east side of Bagerhat in Bangladesh. Table 2 shows the socio-economic conditions of the people studied. Generally socio-economic condition consists of different parameters such as age group, gender distribution, education, housing condition, toilet condition, occupation, income level per month etc. This research reveals that more than half of the respondents have household incomes within Tk 5000 per month, which is below the poverty line. The overwhelming majority of them live in temporary housing systems that are in a poor condition and they also use unsanitary hanging latrines. It can also be observed that almost two-third of the total houses are temporary structures made of bamboo, wood and mud. Such kind of housing are the most vulnerable to the natural hazards.

Table 2: Socio-economic condition of the households selected for the questionnaire survey in two selected sites

Socio-economic parameters	**Population (%) in Morrelgonj**	**Population (%) in Kaliganj**
Age group: Below 15 year 15-60 years Above 60 year	 14.7 63.7 21.6	 31 64 5
Gender distribution: Male Female	 56.9 43.1	 51 49
Education: Illiterate Primary Secondary S.S.C Above S.S.C	 9.8 21.6 11.8 27.5 29.3	 40 23 14 13 10
Housing condition: Kacha Semi-pacca Pacca	 62.7 30.4 6.9	 77 17 6
Toilet condition: Hanging latrine Pit latrine Sanitary latrine	 3.9 49.0 47.1	 38 49 13
Occupation: Business Service holder Farmer Day labor Others	 24.5 24.5 16.7 20.6 13.7	19 13 44 24
Income level per month: 2000-5000 5000-10000 Above 10000	 52.9 30.4 16.7	51 45 4

Unlike for environmentalists and researchers, the coastal community has built a strong perception about the hazard types and dealing with these hazards is an everyday occurrence. During field investigations, people were asked what the common hazard in their community were, the type of changes they have noticed over the decades, and what the factors playing a role in their vulnerabilities were. Table 3 shows their perceptions. It is found that in Morrelganj, about 69% respondents think that the frequency and intensity of hazards is increasing. 88% of respondents gave their opinion in favor of increasing pattern of vulnerability due to hazard. On the other hand in Kaliganj, about 88% respondents think that the frequency and intensity of hazards is increasing. 84% of respondents gave their opinion in favor of increasing pattern of vulnerability due to hazard. Increasing cost of living and increasing rate of population density are found to be most responsible socio-economic factor for increasing vulnerability.

Different parts of coastal areas face different types of hazards, though the common type of hazards in Morrelganj and Kaliganj is shown in Figure 3. 100% people think that cyclone is the most prevalent coastal hazard in both areas. Flood is the second prevalent hazard in Morrelganj and River bank erosion is the second prevalent hazard in Kaliganj. Among others, the tidal surge, riverbank erosion and drought are also the considerable natural hazards. Drought is found to be zero in Kaliganj according to the peoples' perception.

Table 3: Perceptions related to the change in hazard intensity and people's vulnerability

Perceptions related to the change in hazard intensity	Percentage of people's perception in Morrelganj	Percentage of people's perception in Kaliganj
Perception about frequency and intensity of hazards:		
Increasing	69%	88%
Decreasing	31%	12%
Perception about vulnerability to hazards:		
Increasing	88%	84%
Decreasing	12%	16%
Socio-economic factors responsible for vulnerability increase		
Decreasing income	45%	69%
Increasing cost of living	89%	96%
Increasing population density	75%	89%
Crops yield decreasing	47%	69%
Scarcity of job opportunity	42%	63%

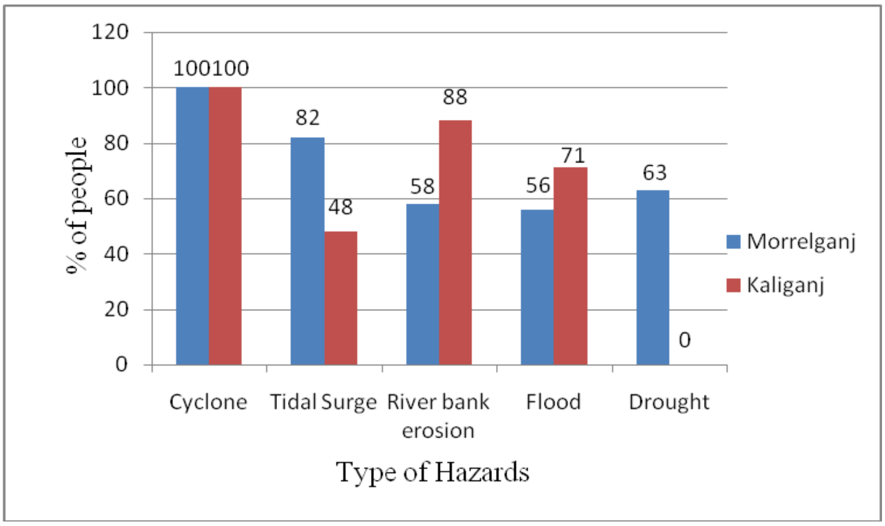
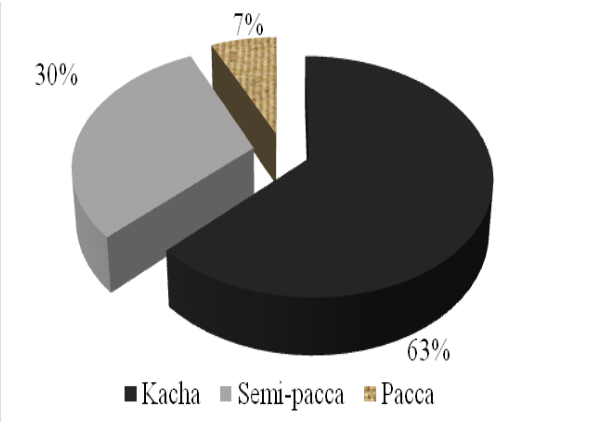
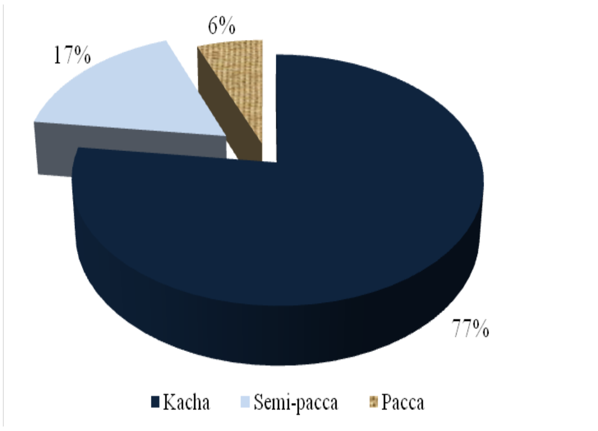


Figure 3: People perceptions of the most prevalent coastal hazards in Morrelganj and Kaliganj.



(a) Existing housing pattern in Morrelganj



(b) Existing housing pattern in Kaliganj

Figure 4: Existing housing pattern in the study areas

3.2 Shelter-Related Impact and Coping Methods for Different Coastal Hazards

Like many other coastal communities, Morrelganj and kaliganj's population also undertakes various coping methods using their abilities, resources and knowledge. Figure 4 shows that, there are about 63% houses in Morrelganj are temporary or kacha whereas about 77% houses in Kaliganj are kacha which are the most vulnerable to coastal hazards.

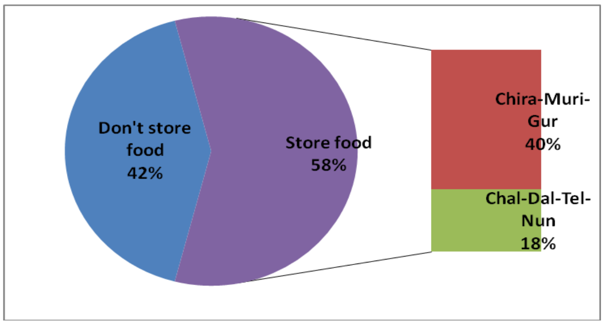
Different coping methods that people follow to protect their shelters and themselves are presented in Table 4. For different hazards people have different coping methods to protect themselves and their belongings. For instance, for different natural hazards, more than two-third of people can't go to cyclone shelter because of large distance from their house. So most of people have to stay in their own house applying their indigenous knowledge or go to their relative's house or neighbor's house.

Table 4: Shelter-related coping methods for different coastal hazards in Morrelganj and Kaliganj

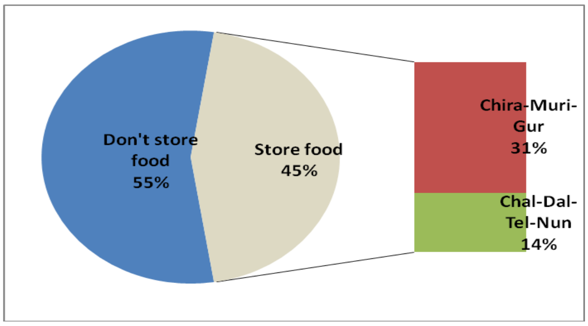
Coping methods	Percentage of people in Morrelganj	Percentage of People in Kaliganj
Self-support using indigenous knowledge	32%	55%
Taking emergency shelter on relative's or neighbor's land	15%	27%
Taking shelter at a cyclone shelter	44%	10%
Others	9%	8%

3.3 Food Crisis during Hazards and Coping Methods

Foods become scarce during every cyclone or other disaster and large amount of households have to suffer for lack of adequate food due to failure of income source.



(a) Storage of food in Morrelganj



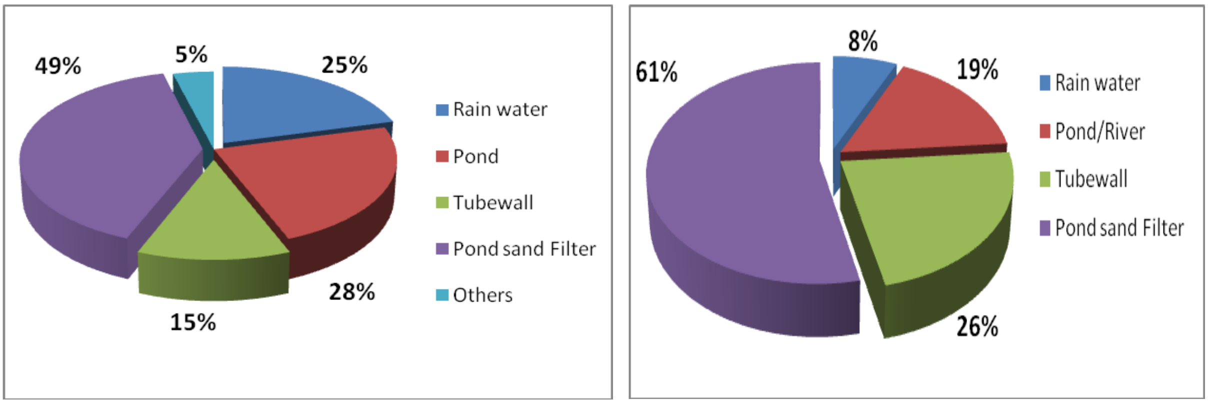
(b) Storage of food in Kaliganj

Figure 5: Distribution of households according to storage of food

During natural hazard expenses for food also increase. Therefore, it deteriorates food consumption, especially for rural poor both in quality and quantity and eventually leads them to the malnutrition and health problem. Field investigation revealed that during natural hazard many people don't store food. Figure 5 shows the distribution of household according to storage of food.

3.4 Impact and Coping Method Related to Water Supply During Natural Hazard

From the Figure 6, it can be realized that most of the people in the area use pond as source of water for drinking and daily activities. The main source of water for drinking and daily activities for these area is tubewell, pond, rain water, pond sand filter etc. Figure 6(a) shows that poeple use tubewell, pond and other source at percentage of 22%, 74% and 4% respectively as their daily activities. on the other hand, Figure 6(b) shows that people use rainwater, pond, tubewell, pond sand filter and other sources at percentage of 25%, 28%, 15%, 49% and 5% respectively as source of drinking water. Generally during natural hazard, the people find the different way for drinking or daily activities due to the unusable condition of the previous water source. Table 5 shows the source of water for drinking and daily activities during and after natural hazards.



(a) Morrelganj (b) Kaliganj

Figure 6: Percentage of usage of source of water for drinking purposes

Table 5: Different sources of water during and after natural hazard

Water supply related coping method during hazard	Percentage of people (%) in Morrelganj	Percentage of people (%) in Kaliganj
Store pure water before hazards	5%	14%
Use water purification tablet	49%	28%
Water boiling	61%	37%
Need to travel long distance	9%	46%

3.5 Impact and Coping Methods Regarding Sanitation

Figure 7 shows the distribution of people according to the usage of different type of toilet. It is found that 47%, 44% and 9% people are generally use sanitary latrine, pit latrine and hanging latrine respectively in Morrelganj. On the other hand, It is found that 27%, 41% and 32% people are generally use sanitary latrine, pit latrine and hanging latrine respectively in Kaliganj. From the field investigation, it can be observed that, maximum toilets become unusable in Morrelganj and Kaliganj. From Figure, It is found that 55% of the total toilets become unusable during natural hazards and only 45% toilets are partially usable in Morrelganj whereas 67% of the total toilets become unusable during natural hazards and only 33% toilets are partially usable in Kaliganj. Figure 8 shows the percentage of peoples' perception related to defecation practices before and during natural hazards.

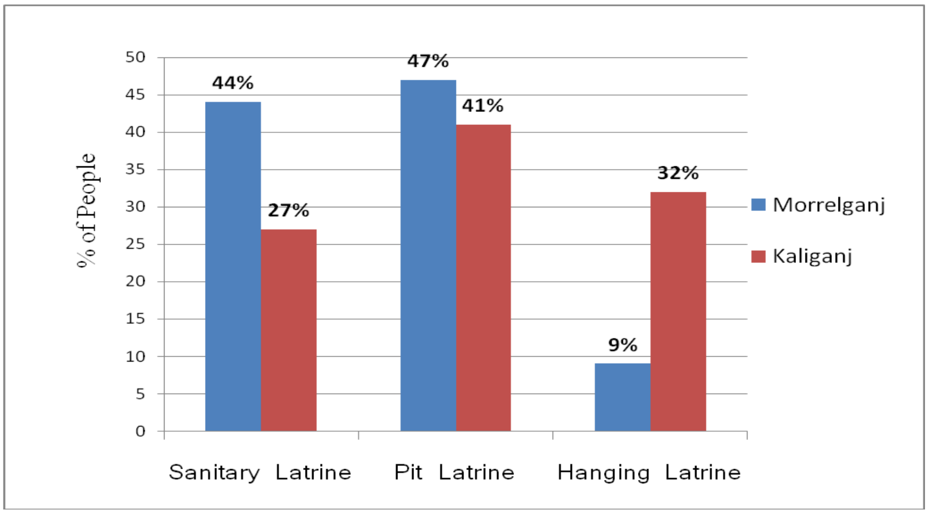
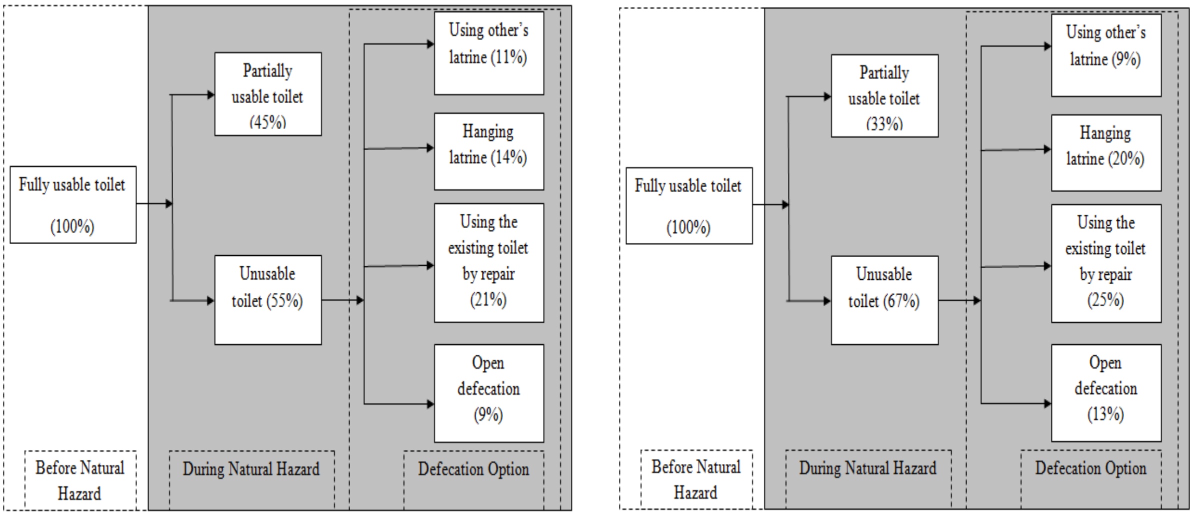


Figure 7: Distribution of people according to the usage of different type of Latrine



(a) Morrelganj (b) Kaliganj

Figure 8: Defecation practices before and during natural hazards

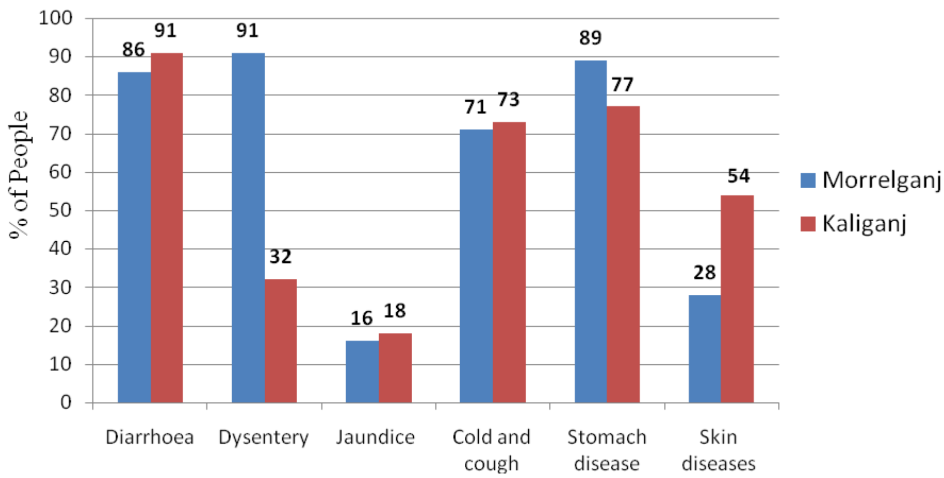
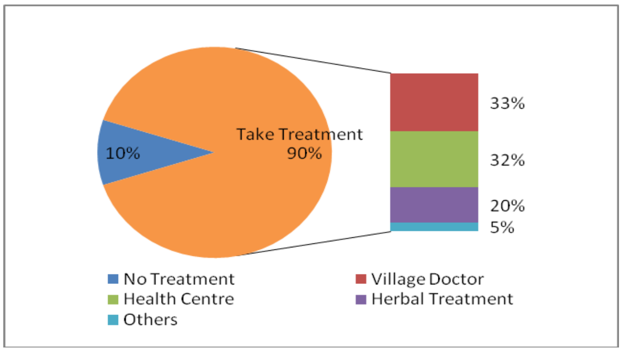


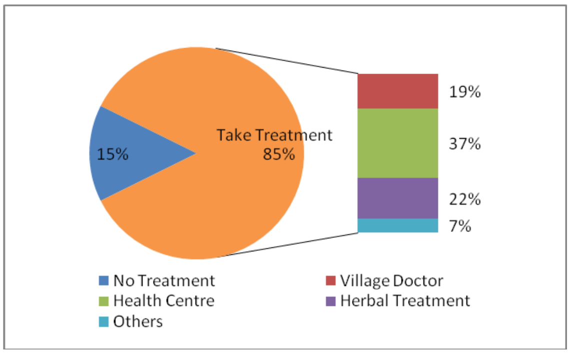
Figure 9: Comparison among diseases according to affected household

3.6 Impact on Health and Coping Method

Figure 9 represents the percentage of people that become affected by different diseases. It is found that the affected people due to diarrhea, dysentery, jaundice, cold fever, stomach disease and skin disease are 86%, 91%, 16%, 71%, 89%, and 28%, respectively in Morrelganj whereas It is found that the affected people due to diarrhea, dysentery, jaundice, cold fever, stomach disease and skin disease are 91%, 32%, 18%, 73%, 77%, and 54%, respectively in Kaliganj. They generally take treatment for the diseases that occurred during cyclone and other natural hazard. Figure 10 shows the distribution of households according to treatment.



(a) Morrelganj



(b) Kaliganj

Figure 10: Distribution of households according to treatment

3.7 Existing Cyclone Shelter in the Study Area

There are two cyclone shelters around the Baloibunia union at Morrelganj. These shelters are not able to accommodate all people of that locality during natural disaster. Figure 11(a) shows one of the cyclone shelters in Baloibunia union at Morrelganj in Bagerhat. On the other hand, there is only one cyclone shelter in Bharashimla union at Kaliganj. Most of the people can't go to the cyclone center because of the long distance from their home. Therefore, they have to stay at their own house to self-support using their indigenous knowledge during natural disaster. They also have to stay at their neighbors' or relatives' house to cope with against the various natural disasters. Figure 11(b) shows the Golkhali Primary School at Kaliganj which is used as a cyclone shelter during natural hazards.

(a) Morrelganj

(b) Kaliganj

Figure 11: Cyclone shelters in the study area

3.8 Existing Water Supply System and Impact during Hazards

Like shelter, people undertake different coping methods to have sage drinking water during hazardous times and to avail treatment in case of sickness. Most of the people from both Morrelganj and Kaliganj use pond or river water for their daily activities, whereas they use pond sand filter made by various Non-Governmental Organization (NGO) for drinking purpose (Figure 12). During disaster, more that 50% water sources are totally unusable and most of the people in Morrelganj use existing pond water by boiling. On the other hand, people need to travel long distance to collect drinking water.

(a) (b)

Figure 12: Pond Sand Filter in (a) Morrelganj (April, 2013) and (b) Kaliganj (August, 2013)

3.9 Existing Sanitation System and Impact during Hazards

Sanitation is one of the important livelihood parameter in a community. From the field investigation, it was observed that most of the toilets that people use are sanitary or pit latrine while few number of people use hanging latrine for sanitation in Morrelganj (Figure 13). On the other hand, about 32% people use hanging latrine in Kaliganj which are connected directly to the water body.

(a) (b)

Figure 13: Existing sanitation system (a) community sanitary toilet in Morrelganj (April, 2013) and (b) hanging toilet in Kaliganj (August, 2013)

CONCLUSIONS

The investigation revealed that both the intensity of coastal hazards and people's vulnerabilities are increasing over time. A number of socio-economic and location factors are enhancing their vulnerabilities though they are relentlessly struggling to minimize their vulnerabilities by undertaking various coping methods. It is also important to note that in different aspect s of life community coping varies with the variation of hazard. The efforts of development organizations need to ensure adequate accommodation in cyclone shelters, which should be connected by a better transportation system and location of the center, should be selected by the community's participation.

According to the socio-economic condition of the both areas, the impact and vulnerability on their livelihood in terms of shelter, water supply, sanitation health and food are increasing day by day. According to the peoples' perception, about 69% people in Morrelganj and 88% people in Kaliganj think that the frequency and intensity of hazards are increasing day by day. On the other hand, about 88% people in Morrelganj and about 84% people in Kaliganj think that the vulnerabilities to hazards are increasing day by day.

According to the peoples' perceptions, the most prevalent coastal hazards in Morrelganj are cyclone (100%), flood (82%), and tidal surge (58%). On the other hand, the most prevalent coastal hazards in Kaliganj are cyclone (100%), riverbank erosion (88%), and flood (71%).

In case of shelter system, there is more kacha house in Kaliganj than Morrelganj. So Kaliganj is more vulnerable than Morrelganj in case of their existing housing pattern. About 44% people go to the nearest cyclone center for emergency shelter in Morrelganj. On the other hand, due to the scarcity of cyclone shelter, about 55% people have to stay their own house and self-support using their indigenous knowledge.

In case of water supply system during natural hazard Morrelganj is more vulnerable than Kaliganj. About 71% water sources are unusable in Morrelganj, whereas, about 55% of water sources become unusable during hazards. Therefore, most of the people in Morrelganj use the flood water by boiling for drinking purpose. Whereas, most of the people need to travel long distance to collect usable water in Kaliganj.

In case of sanitation system, people from Kaliganj use more hanging latrine (about 32%) than Morrelganj. Therefore, latrines in Kaliganj become more unusable than Morrelganj. Thus, sanitation system in Kaliganj is more vulnerable than Morrelganj during disaster.

In case of health and food, people suffered various kinds of diseases due to the impact of natural hazards. Diarrhea is the most prevalent disease in both areas during disaster. Most of the people from both areas take treatment to recover these diseases. They go to village doctor, health center for proper treatment. They also take herbal medicine for recovery. But about 60% people in both areas do not stock medicine in case of emergency condition, even oral saline though most of the people suffered from diarrhea, dysentery, fevers, eye infections, etc. during natural hazard. On the other hand, generally in every locality of Bangladesh as a preparation for natural hazards, households store dry food such as chira-muri, gur (molasses) and chal (rice), dal (pulse), tel (oil), nun (salt) etc. More than half of the people do not store food in Morrelganj and more than half of the people in Kaliganj store food during hazards.

REFERENCES

IPCC (2001). Working Group II Report "Climate Change 2001: Impacts, adaptation, vulnerability". Intergovernmental Panel on Climate Change, Third Assessment Report.

Islam, R.M. (2008). "Towards institutionalization of global ICZM efforts". In: Krishnamurthy RR (ed.) Integrated coastal zone management. Research Publishing Services, Singapore, pp. 23.

Islam, R.M. (2008). "ICZM initiatives and practices in Bangladesh". In: Krishnamurthy RR (ed) Integrated coastal zone management. Research Publishing Services, Singapore, pp. 81-82.

Islam, M.R. (2004). "Where land meets the sea: a profile of the coastal zone of Bangladesh". University Press Limited, Dhaka, pp. 77.

MoWR (2006). "Coastal development strategy". Water Resources Planning Organizations, Ministry of Water Resources, Government of People's Republic of Bangladesh, pp 1, 4-5.

UNISDR (2007). "Disaster occurrence: number of natural disaster registered in EMDAT". Available at http://www.unisdr.org/disaster.statistics/occurrence-trends.centuty.htm.

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DIGITAL ELEVATION BASED FLOOD HAZARD STUDY WITH EFFECT OF CLIMATE CHANGE SCENARIO IN SIRAJGANJ SADAR UPAZILA, BANGLADESH

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ABSTRACT

Sirajganj Sadar Upazila, under the Sirajganj district is surrounded by a network of rivers. Water level and discharge in all these rivers rises in the monsoon (June-September) due to seasonal rainfall and trans-boundary flow which eventually makes the Upazila vulnerable to flooding. In 1997 a Sirajganj Town Protection Embankment was built focused on flood mitigation for the area. But the embankment faces frequently of breaching especially in last few years. The Upazila is very much vulnerable to flooding because of having comparatively low lying areas and deltas and moreover having a big river named Brahmaputra. In the present study, flood hazard zones with and without climate change at various return periods in Sirajganj Sadar Upazila have been identified by simple GIS and Remote Sensing application. The study shows that, significant amount of inundation with various flood depth become high for each land coverage from 5-yr to 100-y return period. The study has considered the influence of the Brahmaputra and Bangali River located at eastern and western side of the study area and for frequency analysis 2.33, 5, 10, 20, 50, 100- year return periods have been chosen.

Keywords: *Inundation, Interpolation, Land use, Return Period*

1. INTRODUCTION

There are two types of floods that occur in Bangladesh: annual floods and low frequency floods of high magnitude. While the annual floods are essential and desirable for overall growth of the Bangladesh delta and the economy, the low frequency floods such as those that occurred in 1954, 1955, 1974, 1984, 1987, 1988, 1993, and 1998 are destructive and cause serious threat to lives (Islam, 1999). Flood hazard assessment is carried out to identify the potential areas of a region for flood mitigation (Tingsanchali & Karim, 2005). After the devastating floods of 1998 in Bangladesh, experts from different fields recommended the need for flood hazard and old maps for the historic events, because of its importance for developing an effective flood management system (Nazneen S, 1998 (as cited in Islam, M.M. 2000)). But for identify the effective approach of flood management by hydraulic structure and land use policy, it is necessary to have a scenario of different flooding extent of both low and high frequencies. In this connection, in this present study, Sirajganj Sadar Upazila under Sirajganj District has been chosen for flood hazard study at various return periods.

Sirajganj is located in north-western zone of Bangladesh and under the district Sirajganj Sadar Upazila with an area of 314.77 sq km is bounded by Raipur Upazila on the north, Belkuchi Upazilas on the south, Kalihati and Bhuapur Upazilas on the east, Kamarkhanda, Raiganj and Dhunat. Upazilas on the west (Figure 1) (Banglapedia, 2008). The area falls in a major Agro Ecological Zones (AEZ), which is the Active Brahmaputra-Jamuna Floodplain (AEZ-8) (Banglapedia, 2008). The main cause of flooding in the area is the transboundary inflow from upstream catchment carried by the Brahmaputra River. It has been estimated that, in the Brahmaputra River, average recorded water level at Bahadurabad ghat, which is an upstream location of the study area is, 15.99 m (PWD). The highest water level recorded at this point is 20.61 m (PWD) on 30, August, 1988 (BWDB, 2009).

The objective of the study is to estimate flood hazard statistics obtained from geoprocessing analysis between inundation maps and land use. In order to achieve the study objective, Flood Frequency Analysis (FFA) of surrounding rivers' water level have been carried out and based on the obtained results inundation maps have been developed and overlaid on major land coverage of the study area. Inundation maps of various return periods depict a picture of water resource management approach at both short and long term. Moreover, the output of this research can also be used as basic information for long-term strategy on disaster reduction. The present study has focused a methodological point of view for identification of flood hazard zones where application of hydrodynamic model would be ambiguous. Otherwise, for riverine flood study, the applied methodology could be an effective tool where, rainfall – runoff and backwater influences will not be considered.

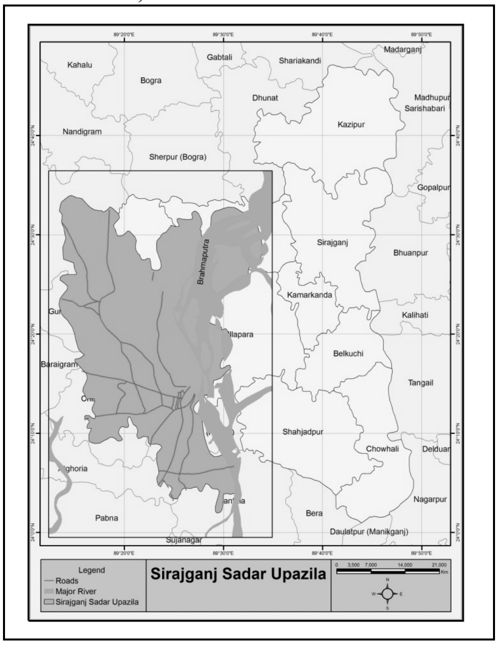

Figure 10: Location of The Study Area

2. METHODOLOGY AND DATA

Satellite Images with the integration of Geographical Information System (GIS) are used for historic flood hazard analysis. National Oceanographic and Atmospheric Administration (NOAA) and Advanced Very High Resolution Radiometer (AVHRR) data were used to analysis Bangladesh's historical flood event of 1988, which sets a hundred - year record for the inundated areas, with severe damage occurring throughout this region (Islam & Sado, 2000). Several hydrodynamic models have also been developed such as HEC – RAS, MIKE11, SOBEK, ISIS, ONDA and FLUCOMP to study inundation at watershed level. Considerable skill is required to determine appropriate cross section locations for such models (Samuels, 1990) and, in addition, bathymetric information with precise resolution, surface nature (topography, vegetation coverage, land use etc) are not explicitly available especially in Bangladesh context.

Apart from hydrodynamic model application for developing inundation maps, Geographic Information System (GIS) Software can also be used. GIS has widely been used to map and model surface water and flood hazard (Aziz et al, 1998 (as cited in (Dewan, A.M. 2004)). Digital Elevation Model (DEM) based flood extent with depth, an integral part of GIS can be adopted for flood hazard study. To get flood map of a study area, flood elevation generated from water level data, is subtracted from ground elevation data (Dewan, et al., 2004). The main disadvantage of the method is unavailability of interpolated water level surface during subtraction with land elevation surface which is interpolated. For obtaining flood extent it is necessary to have both interpolated water level and land elevation surfaces as flooding is a continuous phenomenon so that and interpolation is the procedure of estimating the value of properties at unsampled points or areas using a limited number of sampled observations.

In order to resolve the methodological gap, interpolation technique at GIS system has been applied using water level data of different stations in order to generate interpolated water level surface generation. There are number of interpolation techniques, designed for particular purpose are available in ArcGIS framework. One of is Kriging interpolation, which has been developed based on statistical models that include autocorrelation (ESRI, 2007). But for water level surface generation, the technique will not be appropriate as there is no statistical relation between the different stations in real scenario. On other hand, another interpolation technique, Spline is used for land surfaces generation, as the technique estimates values using a mathematical function that minimizes overall surface curvature, resulting in a smooth surface that passes exactly through the input points (ESRI, 2007). But for interpolating a hydrologically correct surface only Topo to Raster method available in

ArcGIS 9.3.1 is used (ESRI, 2007). In the present study, Topo to raster interpolation tool of ArcGIS has been applied for generating water level interpolated surface generation. The point feature datasets can be converted to 1m resolution ArcGIS grid format datasets using the Topo to Raster tool located in the ArcGIS Toolbox (Tait, et al., 2007). The Topo to Raster tool in ArcGIS 3D analyst results in a connected drainage structure and corrects representation of ridges and streams (Collins, Eric, Forkuo and Mensa, 2012)

Analysis of how often particular flood intensity is likely to occur termed as Flood Frequency Analysis (FFA) is an important concept in flood hazard study. FFA is a technique of statistical examination of the frequency – magnitude relationship (Davie, 2008). It is an attempt to place a probability on the likelihood of a certain event occurring (Davie, 2008). In FFA, return period (T) is used which have a statistical term meaning the chance of excedence once every T years over a long period (Davie, 2008). Inundation maps at different return periods along with adjacent land coverage could be a useful analysis for flood hazard study. In this present study, estimation of water level data collected from three gauge stations SW49, SW66, SW11 of Bangladesh Water Development Board (BWDB) of Brahmaputra-Jamuna , Karatoya and Bangali River (Figure 2) respectively have been used in FFA. The obtained water level results at various return periods have been used as linear interpolation (Topo to ras) with ArcGIS 9.3.1 to develop flood depth maps in different land use classification.

Flood Hydrology Study and Chowdhary and Karim have used different probability distributions and suggested that the Log Pearson type 3 (LP3) distributions are suitable for Bangladesh for the frequency analysis of discharge (Ferdows & Hossain, 2005)[9]. For FFA, commonly used empirical distributions 2- Parameter Log normal (LN2), 3- Parameter Log normal (LN3) Pearson type 3 (P3), Log Pearson type 3 (LP3), Extreme value type 1 (EV1) have been adopted in this study. After that, using Probability Plot Correlation Coefficient (PPCC) and goodness – of – fit test, the best frequency analysis method has been selected for flood mapping in this study. In this study, observed peak water level data collected from Bangladesh Water Development Board (BWDB) have been used for FFA and obtained estimated water level data from the FFA have been used as height source in Topo to Raster interpolation technique for water level surface generation.

Land surface elevation data have been collected from General Bathymetric Chart of Ocean (GEBCO). The GEBCO_08 Grid is a continuous terrain model for ocean and land with a spatial resolution of 30-arc seconds (GEBCO, 2009). Spatially distributed at 50 m interval point dataset has been used for extracting elevation value from GEBCO and water level value from interpolated water level surfaces. Difference between water level data obtained from the interpolated water level surface of different return periods and land surface values has been considered as inundation depth in the study area. Land use or land cover dataset has been generated from the digital image classification of LANDSAT, ETM+ satellite images (Samanta, et al., 2011). Supervised image classification method has been adopted on LANDSAT 7 image downloaded from Global Land Cover Facility web site (Alaguraja, et al., 2010). In this present study, Landsat satellite image has also been used for land use mapping with the help of Integrated Land and Water Information System (ILWIS 3.4) software. Here, only, agricultural, rural settlement, water bodies, urban / growth center and bare soil coverage have been identified. It is necessary to note that, as like as Dewan, et al., 2004, rainfall induced flooding, and / or water logging, inside the study area has not been considered during preparation of flood depths maps. Finally, Inundation layers have been overlaid on land use layer to obtain the overlaid zones termed as flood hazard zones between them. From the ArcGIS overlay analysis different sort of inundation statistics have also been generated and presented graphically.

Increase of flood level and its duration are key factors to characterize the impact of flood due to climate change. It is seen that peak flood level is increased by about 37cm in a moderate flood event (2004 flood event) and in a normal flood event (2005 flood event) the increase is 27cm in the Jamuna river (IWM, 2008). More specifically, in Sirajganj maximum flood depth would be increased by 36cm in 2040 considering the flood events of 2004 as IWM suggest. Another study conducted by IWFM and CEGIS in Collaborative Research Project 'CLASIC' suggested that, 16.26 % flow would be increased in the Brahmaputra River by 2050 for B1 scenario (IWFM, CEGIS, 2008) which is based on environmentally sustainable economic growth, decline population growth after mid-century. Therefore, flood extent would also be increased by 2.08% due to climate change condition as suggested by IWM, 2008. In this connection the present study shows flood hazard map due to climate change condition for Sirajganj sadar upazila for year of 2040. The secondary above studies consider the base condition of flood year is 2004 which is a 50 year return period flood event for predicting climate change scenario, therefore, 36cm flood level rise has been considered for Sirajganj sadar upazila flood hazard mapping.

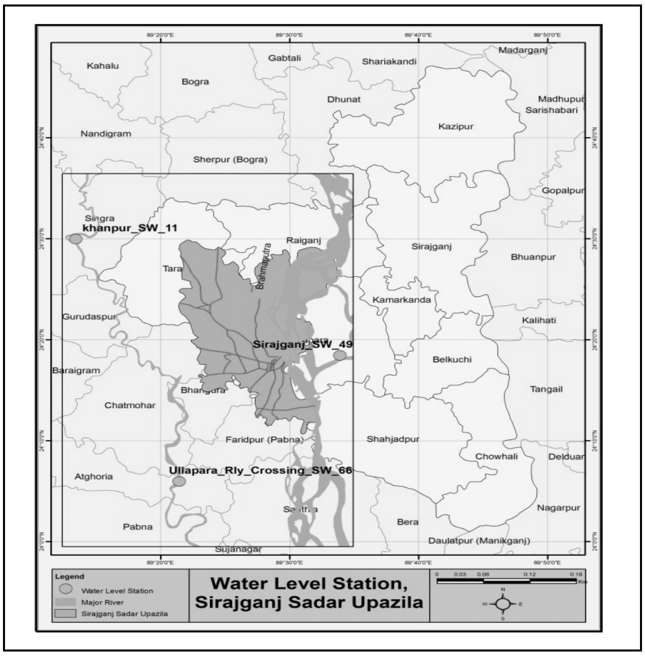

Figure 2: Water Level Station, Sirajganj Sadar Upazila

3. RESULTS AND DISCUSSION

3.1 Land use of the study area

Supervised classification of landsat image with ILWIS 3.4 software depicts different land coverage existing in the study area. To have training areas, ground truthing points collected from field survey have been used for this image. classification. From the land use map, others classification indicates different sorts of uneven phenomena in the study area. An uneven phenomenon includes bare soil, bush land, brick fields, open spaces etc. About 28% and 13% area is covered by agricultural and rural settlement area respectively. Urban percentage is negligible, amounted to 1.90% of the total area (Figure 3).

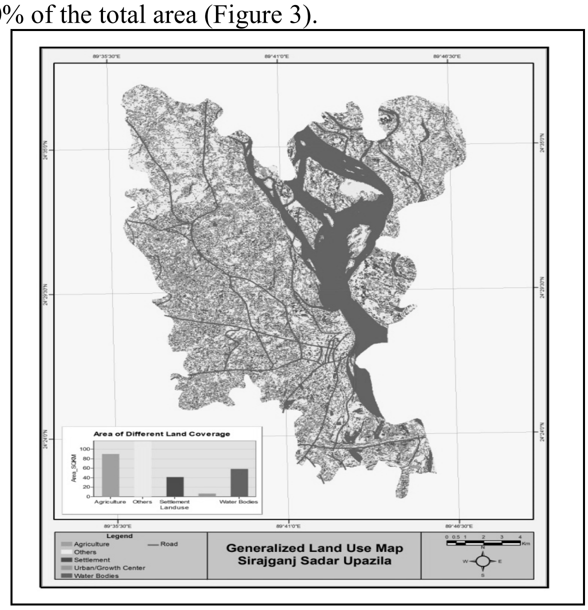

Figure 3: Generalized Land Use Map

3.2 Flood Frequency Analysis Inundation Maps

In the present study, the regression test for linear trend has been carried out for the annual water level series from 1981 to 2002 at Khanpur (SW 11) station of Bangali River, Sirajganj (SW 49) station of Brahmaputra -

Jamuna River and Ulapara Railcrossing (SW 66) of Deonai River. The graphical trend line of peak water level for all three stations has been presented in Figure 4, Figure 5 and Figure 6.

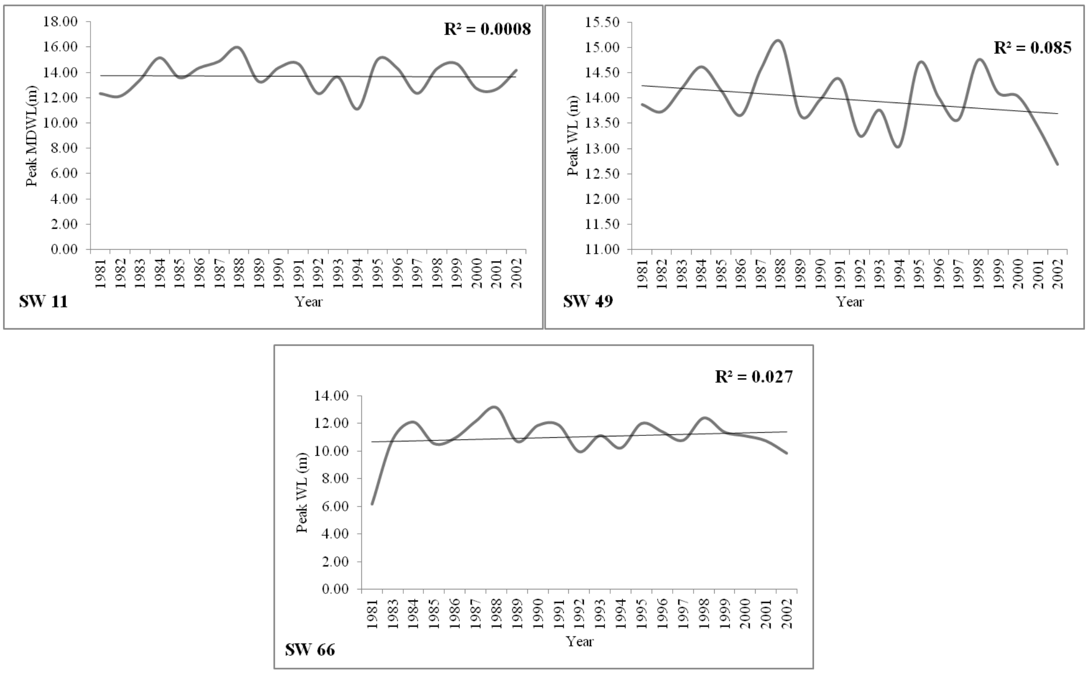

Figure 11, 5, 6 : Trend of yearly maximum water level at three stations

After that, the trend free hydrologic data have been used to determine designed flood levels for several return periods (2.33yr, 05yr, 10yr, 20yr, 50yr and 100yr floods) through flood frequency analysis. For selecting best fitted distribution, goodness-of-fit test has been conducted. In the goodness-of-fit test, Probability Plot Correlation Coefficient (PPCC) had been applied. The results of the test are shown in Table 1, Table 2 and Table 3. Based on the result of the goodness-of-fit test, Pearson type 3 (P3) has been selected for SW 11 and SW 49 and Log Pearson type 3 (LP3) has been selected for SW 66 station for flood frequency analysis. Figure 7, Figure 8 and Figure 9 shows the plotted results at station SW11, station SW49 and station SW66, respectively.

Table 1 Goodness-of-fit test for selecting most appropriate distribution, SW11

	Station: SW11							
PDF	Return Period						PPCC	Rank
	2.33 yr	5 yr	10 yr	20 yr	50 yr	100 yr		
LN2	13.99 m	14.89 m	15.53 m	16.08 m	16.37 m	17.16 m	0.97801	4
LN3	13.99 m	14.72 m	15.18 m	15.53 m	15.71 m	16.14 m	0.98581	2
P3	13.96 m	14.72 m	15.20 m	15.59 m	15.79 m	16.28 m	0.98593	1
LP3	14.08 m	14.91 m	15.44 m	15.87 m	16.08 m	16.62 m	0.98550	3
EV1	13.69 m	14.56 m	15.27 m	15.96 m	16.35 m	17.50 m	0.94884	5

Table 2 Goodness-of-fit test for selecting most appropriate distribution, SW49

Station: SW49								
PDF	Return Period						PPCC	Rank
	2.33 yr	5 yr	10 yr	20 yr	50 yr	100 yr		
LN2	14.15 m	14.52 m	14.77 m	14.99 m	15.10 m	15.40 m	0.99260	2
LN3	14.13 m	14.48 m	14.72 m	14.92 m	15.02 m	15.30 m	0.99249	3
P3	14.14 m	14.48 m	14.71 m	14.90 m	15.00 m	15.26 m	0.99321	1
LP3	14.13 m	14.51 m	14.79 m	15.03 m	15.15 m	15.50 m	0.98986	4
EV1	14.05 m	14.42 m	14.72 m	15.01 m	15.17 m	15.66 m	0.97163	5

Table 3 Goodness-of-fit test for selecting most appropriate distribution, SW66

Station: SW66								
PDF	Return Period						PPCC	Rank
	2.33 yr	5 yr	10 yr	20 yr	50 yr	100 yr		
LN2	11.71 m	12.28 m	12.68 m	13.02 m	13.41 m	13.68 m	0.991416	3
LN3	11.67 m	12.27 m	12.71 m	13.11 m	13.58 m	13.91 m	0.990575	4
P3	11.71 m	12.27 m	12.71 m	13.03 m	13.43 m	13.71 m	0.99158	2
LP3	11.70 m	12.28 m	12.69 m	13.05 m	13.47 m	13.76 m	0.991586	1
EV1	11.60 m	12.20 m	12.69 m	13.16 m	13.76 m	14.22 m	0.979099	5

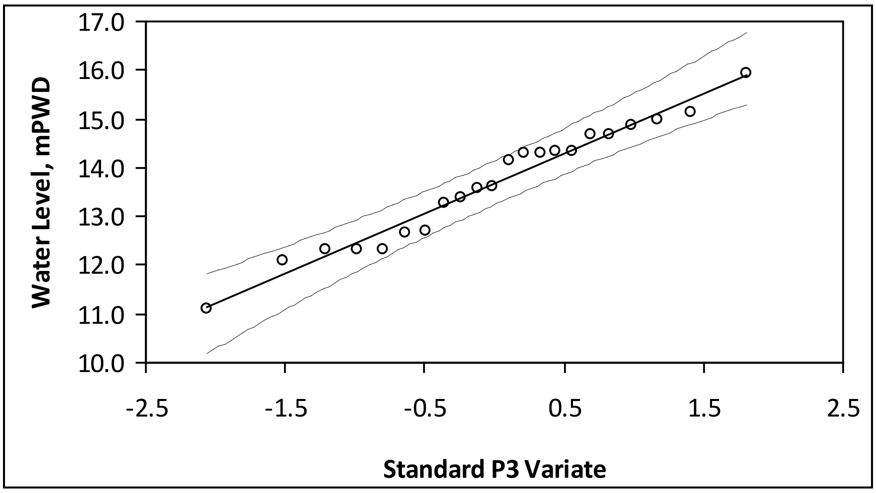

Figure 7 Flood frequency analysis at station SW11

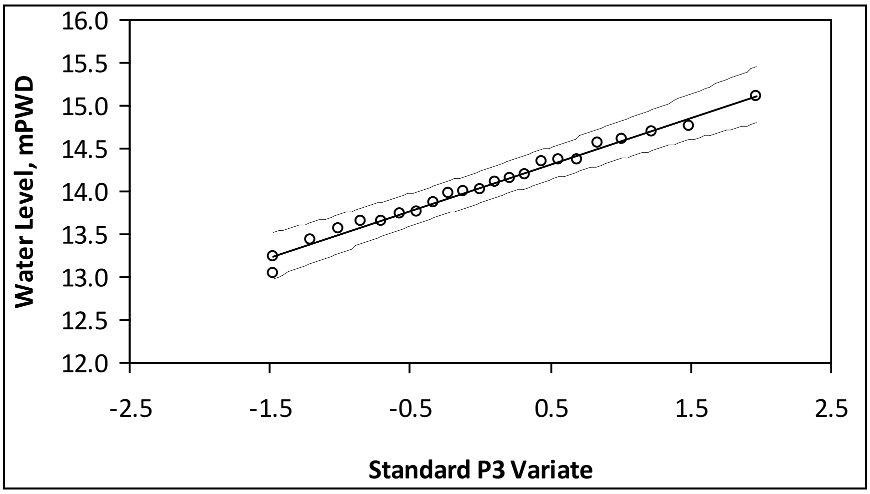

Figure 8 Flood frequency analysis at station SW49

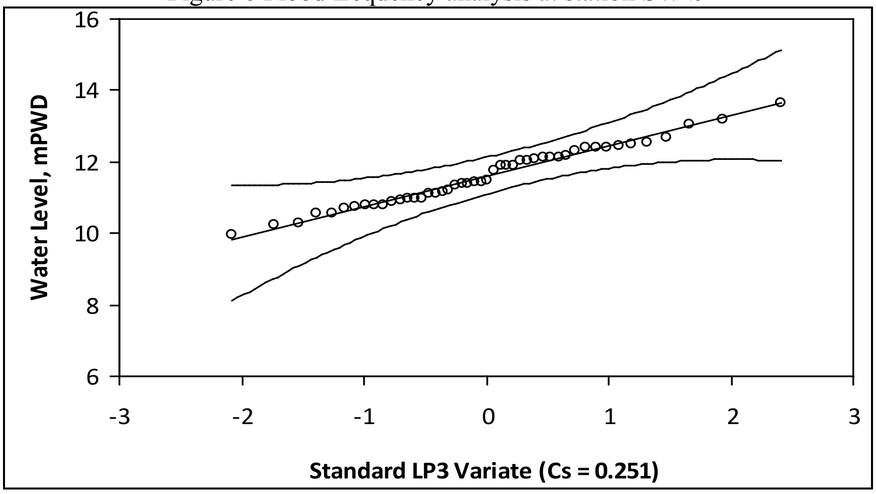

Figure 9 Flood frequency analysis at station SW66

3.3 Inundation Maps

Figure 11 depicts the general behavor of topography generated from GEBCO data set. The flood water levels obtained from FFA for various return periods have been used for Topo to ras interpolation technique and overlain onto the land surface elevation of the study area. Different between water level interpolation and land elevation surfaces has been considered as a depth of inundation for each return period (Figure 12). Figure 13 depicts area inundated by floods of different return periods and it shows that, with the increase of return period the inundated area increases substantially for the flood class of Low (0.1 – 2.0 m). For Medium (2.1 – 5.0 m) and High (5.1 – 10.0 m) class of flood, inundated area also increases modelrately but it remains almost same for Very High (10m+) class of flood. On the other hand, flood free zone decrease with the increase of return period. Study conducted by (Institute of Water Modeling, 2010) termed the 1998 flood is a return period of 75 to 100 year. The percentage area of flooded during 1998 in Sirajganj sadar upazila was 54 as on September, 17 (BWDB, 2010). In this study, for 100 year return period, 46.10 (144.98 Sq. Km) percent area of flooding has been found, which is quite close to BWDB study (Figure 10).

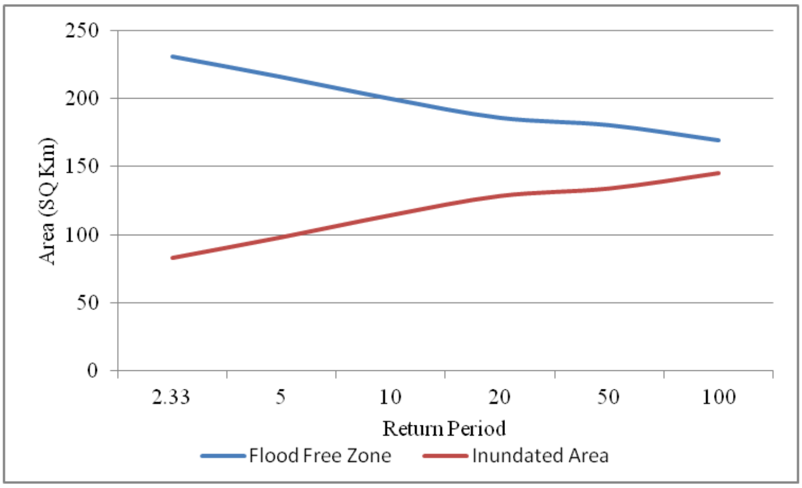

Figure 10 Inundated Area and Flood Free Zone at Various Return Periods

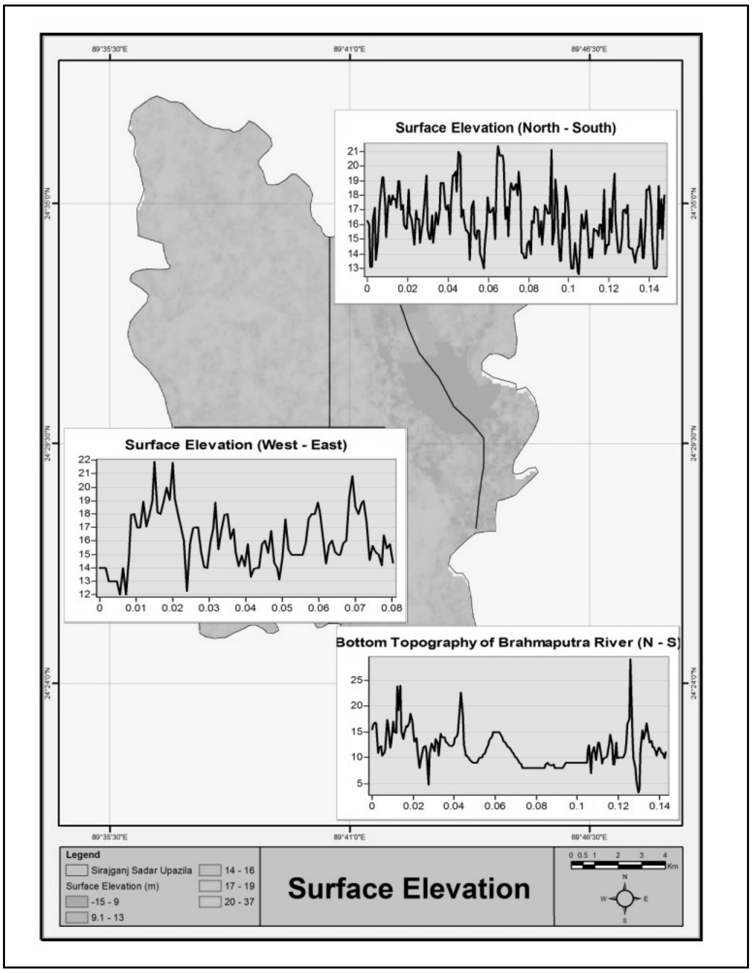

Figure 11 Surface and Bottom Topography

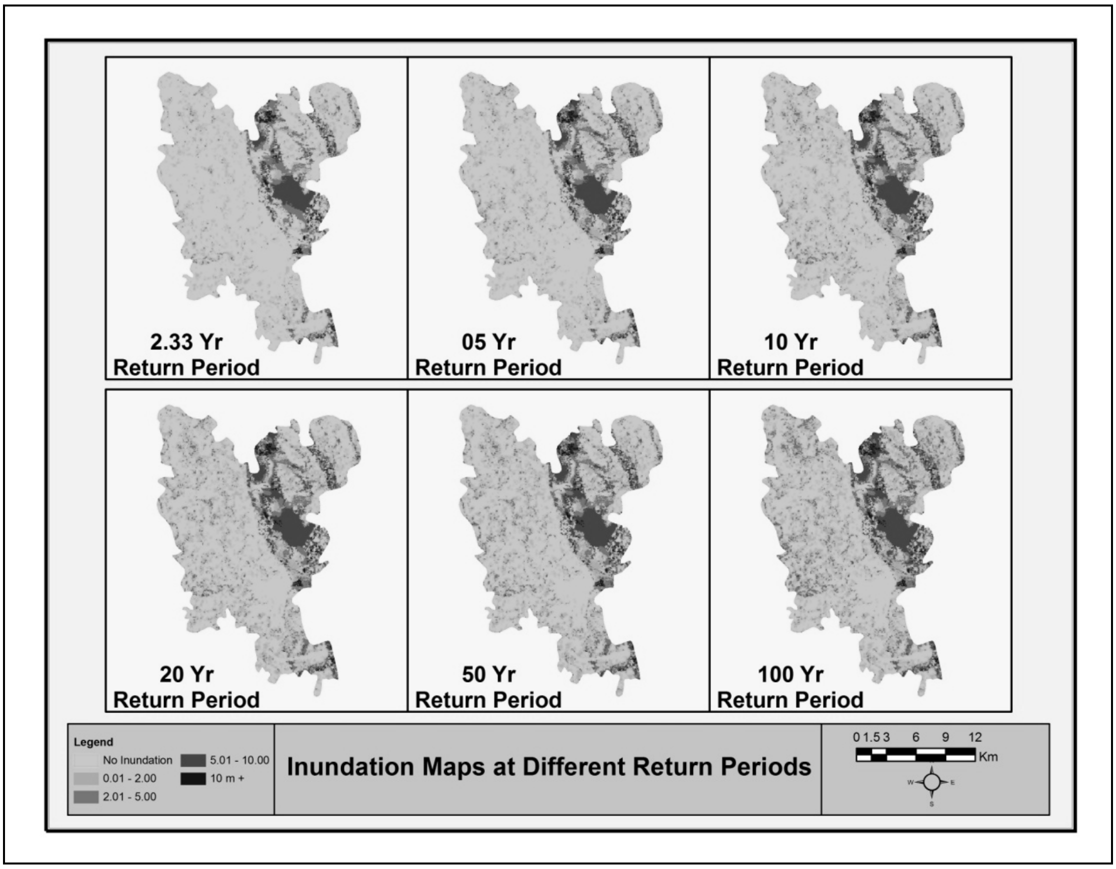

Figure 12 Inundation Maps at Different Return Periods

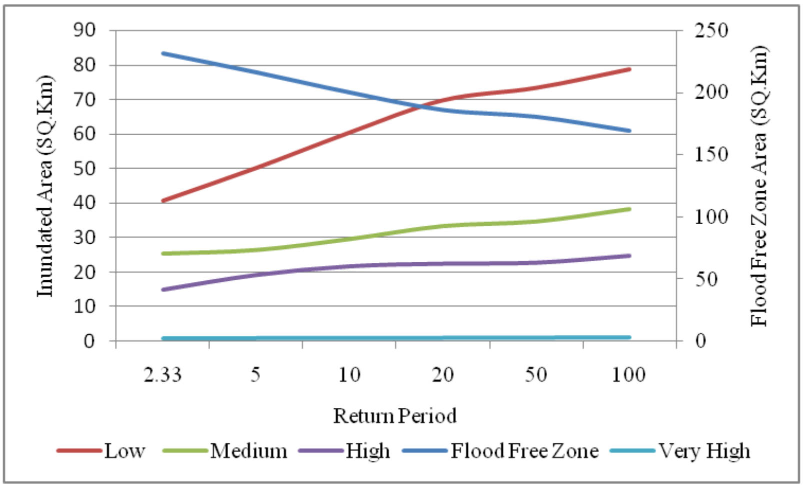

Figure 13 Area inundated in various flood classes for different return period

3.4 Flood Hazard at Different Return Periods

For all land use classes, affected area increases with the increase of return periods and flood depth. It is also noticeably that, inundated areas become doubled for land use classes agricultural and rural settlement with the increase of return periods. Rural settlement areas are inundated much more than that of urban settlement with the increase of return period and flood depth. Water bodies including existing rivers are become inundated much more than that of any other classes of land use especially for medium to very high classes of flood. It reflects the captured fisheries loss during floods. Agriculture, which is the most dominant use of land in the study area, rising trend of inundation become decreases with the increase of flood depth. Noticeably, in case of high and very high class of floods, inundated area for agricultural use remain same with the increase of return period. Bare soil, mostly occupied at deltaic lands in the rivers are highly prone to floods particularly for low and medium classes of floods. Figure 14 and 15 depicts the inundation statistics and inundated area in percentage respectively.

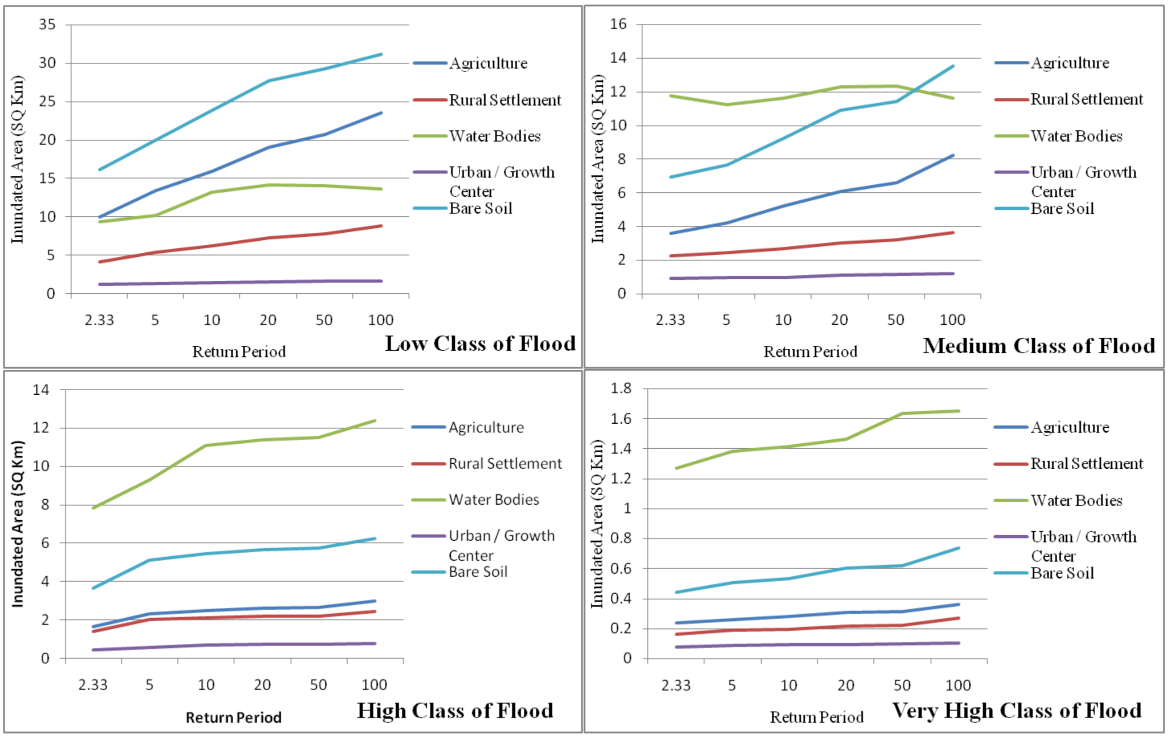

Figure 14 Inundation statistics for different return periods and flood classes

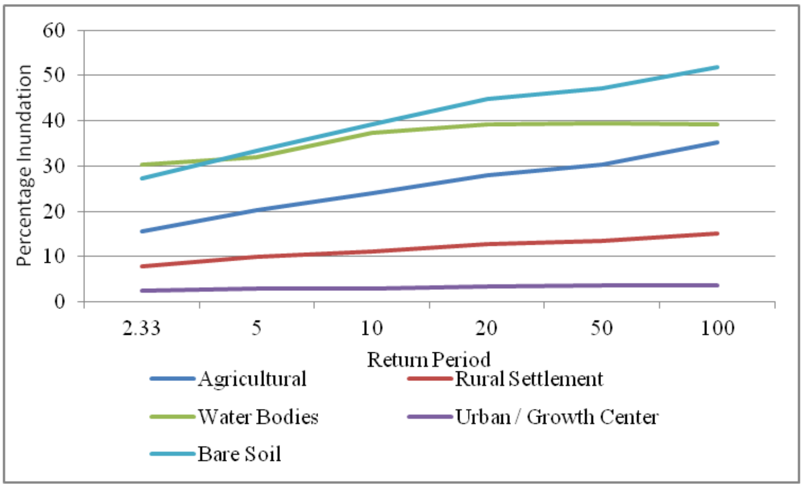

Figure 15 Inundated area (%) for each land use classes at different return periods

3.5 Flood Hazard with and without Climate Change

Bangladesh is extremely vulnerable to climate change because of its geophysical settings. It is a low-laying flat country with big inland water bodies, including some of the biggest rivers in the world. Flooding is an annual recurring event during monsoon and 80% of annual rainfall occurs in monsoon. If rainfall increases due to climate change in the GBM basin that will create huge water flow through the rivers of Bangladesh. Eventually the monsoon flood will be more devastating due to increase of precipitation and sea level rise that may cause more damage to crops and properties if adaptation measures are not taken. Hazard maps with and without the affect of climate change are shown in (Figure 16).

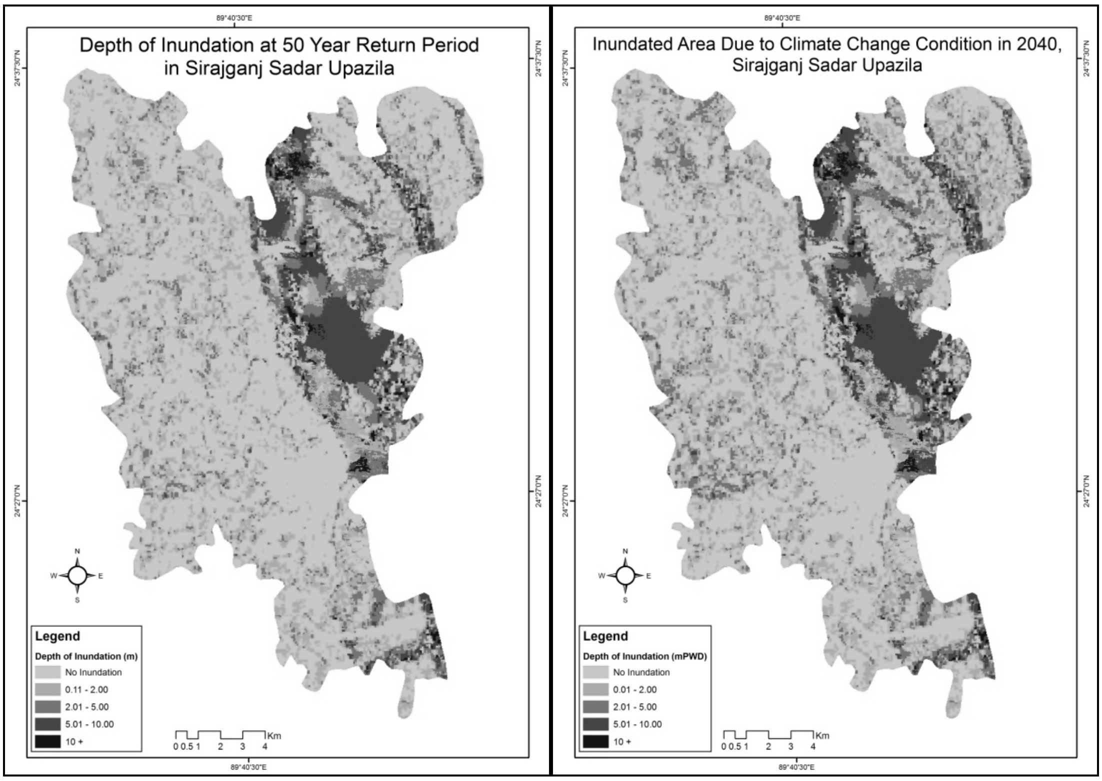

Figure 16 Inundated Area with and without Climate Change Scenario in Sirajganj Sadar Upazila, 2040

Table 8.13 Inundation Statistics with and without Climate Change Scenario in Sirajganj Sadar Upazila, 2040

Inundation Extent	Crop land	Rural Settlement	Urban/Growth Center	Others
Without Climate Change (sq.km) Total (133.84)	30.33	13.36	3.56	86.56
With Climate Change (sq.km) Total (157.82)	38.31	16.49	3.92	88.93
Percentage Increase (17.91)	26.31	23.42	10.11	2.73

4. CONCLUSIONS

The present research could be a good and an effective methodological study in river flood hazard study whereas, hydrodynamic model application might be quite ambiguous. Moreover, it can be said that, in order to manage floods in the study area, hydraulic structures should be designed on the basis of result findings. In addition, land use policy might be adopted on basis of the study result. Special attention should be drawn for urban flood management of the city. So, the planners and the decision makers may find the result of the study useful for framing an appropriate flood risk management plan in the Sirajganj Sadar Upazila.

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REFERENCES

Alaguraja, P. et al., 2010. Land Use and Land Cover Mapping – Madurai District, Tamilnadu, India Using Remote Sensing and GIS Techniques. International Journal of Civil and Structural Engineering, 1(1), pp. 91 - 100.

Banglapedia, 2008. Dhaka: Bangladesh Asiatic Society.

BWDB, 2009. Non- Tidal Water Level. Dhaka: Bangladesh Water Development Board, Ministry of Water Resources.

BWDB, 2010. Flood 1998. [Online] Available at: http://www.bwdb.gov.bd/index.php?option=com_content&view=article&id=196&Itemid=198 [Accessed 5 May 2013].

IWM, 2008. Climate Change Prediction Modeling Impact Assessment of Climate Change and Sea Level Raise on Monsoon Flooding.

IWFM, CEGIS, 2008. Impact of Climate Change on Surface Water flow in Bangladesh.

Collins, Eric, Forkuo and Mensa, 2012. River Inundation and Hazard Mapping – a Case Study of Susan River – Kumasi. s.l., GSDIA.

Davie, T., 2008. Fundamentals of Hydrology. 2nd ed. New York: Routledge.

Dewan, A., Nishigaki, M. & Komatsq, M., 2004. DEM Based Flood Extent Delineation in Dhaka City, Bangladesh. Journal of the Faculty of Environmental Science and Technology, 9(1), pp. 99 - 110.

ESRI, 2007. Introduction to ArcGIS Spatial Analyst. s.l.:Environmental System Research Institute.

Ferdows, M. & Hossain, M., 2005. Flood Frequency Analysis at Different Rivers in Bangladesh: A Comparison Study on Probability Distribution Functions. International Journal of Science and Technology, 10(3), pp. 53 - 62.

GEBCO, 2009. The GEBCO_08 Grid. [Online] Available at: www.gebco.net/data_and_products/gridded...data/.../gebco_08.pdf [Accessed 2013].

Institute of Water Modeling, 2010. Report on Flood Hazard Model for Sirajganj District, Dhaka: IWM.

Islam, M. & Sado, K., 2000. Flood hazard assessment in Bangladesh using NOAA AVHRR data with geographical information system. Hydrological Processes, Volume 14, pp. 605 - 620.

Islam, N. C. f. U. S. D. 1. p., 1999. Flood 98 and the future of urban settlements in Bangladesh. Flood and Urban settlements, p. 19.

Samanta, S., Pal, D., Lohar, D. & Pal, B., 2011. Preparation of Digital Data Sets on Land use/land cover, Soil and Digital Elevation Model for Temperature. Indian Journal of Science and Technology, 4(6), pp. 636 - 642.

Samuels, P., 1990. Cross section location in one-dimensional models. Chichester, Wiley, p. 339–350.

Tait, Sturman & Smart, 2007. Inundation mapping of future high tides, sea level rise and storm surge.

Tingsanchali, T. & Karim, M., 2005. Flood hazard and risk analysis in the southwest region. Hydrological Processes, Volume 19, p. 2055–2069.

Myeong, S, Hyun, J. – Developing Flood Vulnerability Map For north Korea, ASPRS 2009 Annual Conference, Baltimore, Maryland. March 9-13, 2009

Proceedings of the 2nd International Conference on Civil Engineering for Sustainable Development (ICCESD-2014), 14~16 February 2014, KUET, Khulna, Bangladesh, ISBN: 978-984-33-6373-2 (CD-ROM)

NATURAL DISASTERS IMPACT ON THE WATER CYCLE, RESOURCES, QUALITY AND HUMAN HEALTH

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ABSTRACT

Bangladesh is facing the threat of global warming and sea level rise. This impact is severe in the coastal region. Proper water resources management is must in this problematic area. The coastal belt of Bangladesh is under a polder system. This system protects the mass people from great threats of natural and man maid calamities over the time. Human requirement of water is increasing day by day as life is getting better and better. So, a good management is required to augment the water resources. This research is aimed at filling the gaps in existing knowledge and management of Bangladesh's surface water resources. One of the challenges facing water managers understands how changing land use and land cover, evaporation and floods impact surface water resources. We are developing tools and methods to manage the increased pressure on water resources from different users and the competition for water between competing uses including interception from vegetation (such as plantation forestry or farming). Surface water resources are replenished by floodplain inundation and groundwater recharge from landscapes, but these events are episodic so the dynamics of the water balance need to be understood to fully appreciate the implications of variable and changing climate.

Keywords: *Bangladesh Natural Disasters, WR Engineering, Disaster Management*

1. INTRODUCTION

Bangladesh is in the permanent stage of disaster due to its geographical position. Being the greatest delta in the world, Bangladesh faces natural disaster like floods, cyclones, river erosion every year. Bangladesh is prone to floods, the most recent flood takes place in Bangladesh is result of cyclone Sidr (November 15, 2007). It was cyclonic storm that brings havoc in southern and central Bangladesh. It killed almost 4000 people and injured about 30.000. Unofficial sources say that the numbers are even higher. The aftermath of the cyclone was more devastating. Sidr lefts flood to the country that took 60,000 houses. But this all was not our fault, almost 3 million people were evacuated from the cyclonic zone but the result was still beyond the imagination. The secondary impacts of the cyclone or the devastating flood is the bare land,trees were uprooted, crops were destroyed, entire roads and bridges were totally washed away, telephone poles were smashed to the ground, electricity was cut off and livestock was killed. This situation tremendously affects our already fragile economy. Specialists blame deforestation and climate change as the main actors for frequent flooding. Climate change causes the sea water levels to rise, meaning that the coast of Bangladesh will narrow each year and will get flooded more often.The most frequent natural disaster harming our resource quality, health system and misbalancing water cycle.

2. NATURAL DISASTERS IN KHULNA

Khulna is generally subjected to natural hazards such as flash flood, cyclonic storm surges and tornado almost every year (Fig 2.1). In addition, the region has severe constraints due to certain unfavorable soil and land qualities such as salinity and water-logging. Also included are a plethora of hydro-geo-morphological hazards which include poor drainage through its river systems, high rates of sedimentation on river beds, acute low flow conditions during the dry season, salinity ingress along the rivers, moisture stress in the dry season, rise in sea level, and to a lesser extent, and flood. While Chittagong located in the coastal zone and Khulna lying in the exposed coastal zones of the country, these regions are significantly influenced by tidal effects.

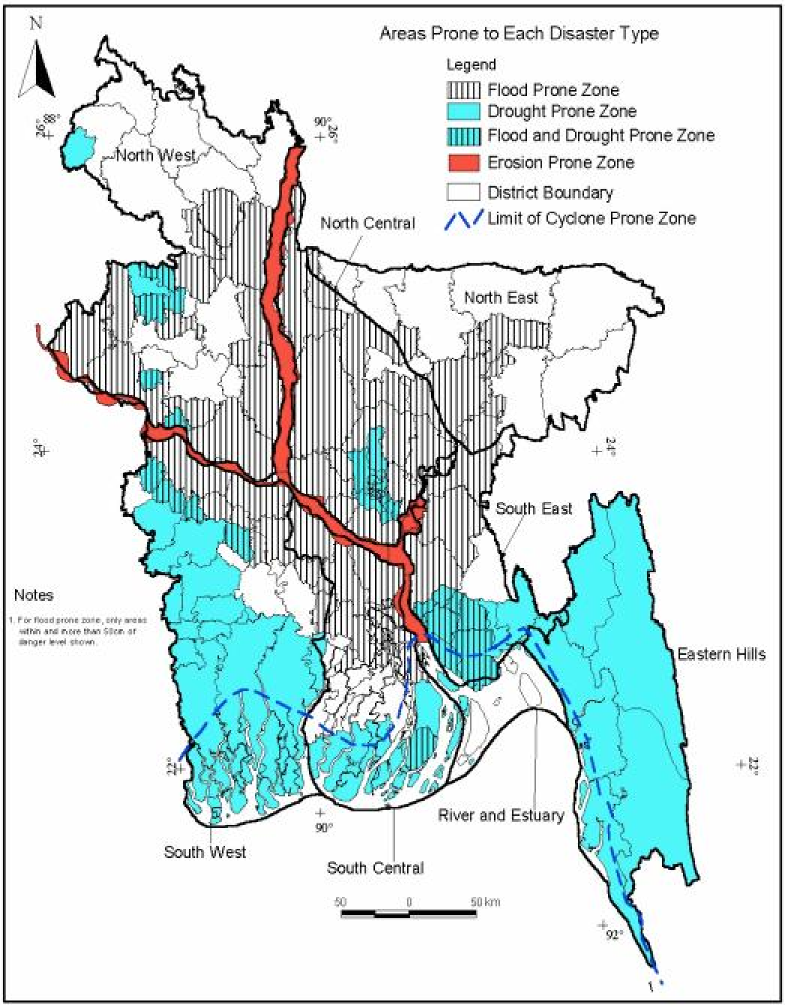

Fig 2.1: Natural hazard areas in Bangladesh.

2.1 Natural Disasters in Bangladesh

Natural calamity means natural disaster which is caused by nature.e.g.

- ✓ Floods
- ✓ Cyclones
- ✓ Tsunami
- ✓ Earthquake
- ✓ River erosions
- ✓ Drought etc.

2.2 Water Cycle

The water cycle is called the hydrologic cycle. In the hydrologic cycle, water from oceans, lakes, swamps, rivers, plants, can turn into water vapor. Water vapor condenses into millions of tiny droplets that form clouds. Clouds lose their water as rain or snow, which is called precipitation. Precipitation is either absorbed into the ground or runs off into rivers. Water that was absorbed into the ground is taken up by plants. Plants lose water from their surfaces as vapor back into the atmosphere. Water that runs off into rivers flows into ponds, lakes, or oceans where it evaporates back into the atmosphere.

2.3 Problems

Bangladesh is the lowest riparian of three major river system of south Asia namely the Ganges –padma, the Brahmaputra, jamuna and the meghna-barak. Another feature is the funnel shaped Bay of Bengal in the south part creates the land as the meeting place of monsoon rains. The land of Bangladesh is flat, with some up-lands in the northeast and the southeast. The great plain lies almost at sea level along the southern part of the country and raises gradually towards the north. Land elevation in the plain varies from 1 to 90 meters above the mean sea level. The maximum elevation is 1231 m in hill district.this information surely showed up the reasons behind further natural calamities. The main target of this study is to find out the reasons, loss and damage assessment for natural disaster, especially flood in Khulna region.

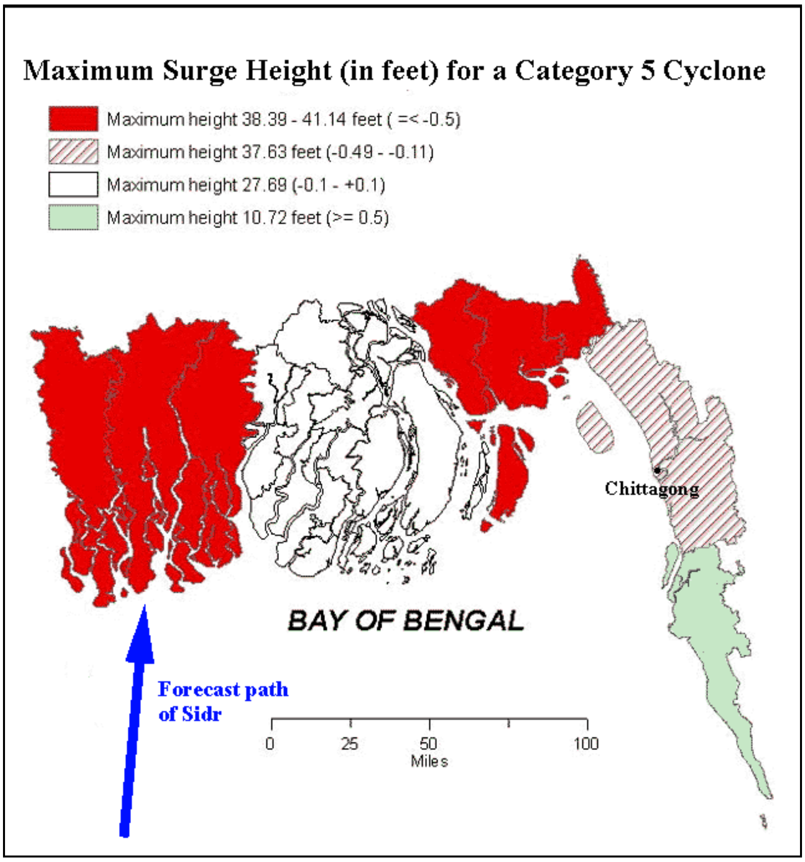

Fig 2.2: Cyclonic storm surges

According to available statistics on Coastal Zone, majority of land is within one meter from mean sea level, a significant proportion of which again falls below high-tide level (Fig 2.2). Over the past few years, natural disasters in this south eastern region have become more frequent and devastating. After the flood in 1998, there was another inundation in 2004 that flooded about 2/3rd of the country including the dry & drought prone south-eastern regions and affected in total more than 30 million people and destroyed around 2 million acres of crop land (www.dmb.com). In October 2008, the southwest coast was hit by Cyclone Rashmi, and in November of the same year Cyclone Sidr (Fig 2.3), a category four super cyclone, hit the south and south-west coast causing extensive damage and the loss of over 4,000 lives. Cyclone Sidr devastated around 4 million families and displaced 6 million people.

Fig 2.3: Cyclone Aila washed the some of Khulna district area with saline water.

In 2009, cyclone Aila , the category 1 cyclone, hit South-Eastern coastal region of Bangladesh on 25 May 2009 and affected 1 million people, displaced around 2 lakh, damaged embankment and 6393 acres of crop fields with saline water (WorldWide, DanChurchAid, MuslimAid, Relief, Oxfam-GB, & Children-UK, 2009). Cyclone Aila affected an estimated 3.90 million people in 11 coastal districts of the Bangladesh's 64 districts. About 2.3 million people were affected by Aila and many of them stranded in flooded villages as they had no alternative to save themselves. Even though Aila was a weak category cyclone by definition, its economic cost outweighs the impacts of Super cyclone Sidr and brought in long-term sufferings for the southeastern people of Bangladesh. The impact was aggravated as the cyclone hit Bangladesh during the high tide cycle that resulted to tidal surges of up to 22 feet. The surge of water caused portions of the embankments to collapse and people who believed that the embankments could protect them did not have enough time to evacuate to higher and safer ground. During Cyclone Aila, the storm spent more time over-land than Cyclone SIDR in 2007, lingering over the coast of Bangladesh and increasing its impact on the vulnerable villages. Over 50% of displaced people (more than 200,000) are still living in the same condition in severely affected Khulna and Shatkhira District (Dasgupta, et al., 2011). The devastations left by Cyclone Aila still remain chronic till date jeopardizing the livelihood patterns and settlements in the area.

3. EFFECTS OF LACK OF SAFE DRINKING WATER

Fig 3.1: long line for safe water.

The effects of scarcity of safe drinking water in south-eastern region can have health, social and financial implication. People in the region suffer from various diseases caused by drinking an insufficient amount of water and drinking water with high levels of salinity, impurity or arsenic contamination. Various skin diseases, intestinal diseases, dysentery, fever and diarrhea are part of life. Other health concerns linked to a lack of safe drinking water include malnutrition amongst women and children, reproductive problems for pregnant women, skin turning black, physical weakness and anxiety. (Fig 3.1)

Women and girls face a number of rights abuses as a direct result of the lack of safe drinking water. in rural bangladesh it is the women's role to collect drinking water. The drinking water can be many kilometers from the home and there are frequent incidents of violence against women and girls for not fetching drinking water on time or not having meals prepared because of the amount of time it takes to fetch water.

Fetching water means women do not have time to tend to their homestead garden, which is often their only source of productivity and income. there are other social crises associated with poor access to safe drinking water: the education of children is hampered; young children are often left unattended when their mother goes to fetch water; they are frequent incidents of child labor; the household has less time to socialise and develop social networks; women are teased and harassed on their way to fetch water; social stigma prevents girls getting married and leads to an increased rate of divorce; population migration; and local contentions and litigations related to water use have become a regular phenomena.gathering drinking water means a significant amount of productive hours is consumed. Household expenditure increases to purchase fresh water to enable cultivation of crops. Cost of buying vegetables increases whilst the durability of houses is reduced and scarcity of food occurs. Maintaining livestock and poultry become difficult. Scarcity of organic fertilizer makes carrying out agricultural activities difficult. All these factors together constitute a major economic problem for the poor.

4. STUDY AREA

4.1 About khulna

Khulna is located in south-western Bangladesh with a total area of 59.57 km², while the district itself is about 4394.46 km². It lies south of Jessore and Narail, East of Sathkhira, West of Bagerhat and North of the Bay of Bengal. It is part of the largest delta in the world. In the southern part of the delta lies the Sundarban, the world's largest mangrove forest. The city of Khulna is in the northern part of the district, and is mainly an expansion of trade centers close to the Rupsha and Bhairob rivers.Like the other big cities of Bangladesh, notably Dhaka and Chittagong, Khulna is undergoing a major transformation, due to its immensely growing population and its status as Bangladesh's third largest city. Because of its strategic location of only 45 km from the port Mongla, Khulna is considered as being a port city like Chittagong. 25% of all trade handled in Bangladesh passes through Mongla, while the rest goes through Chittagong[2]. Khulna is also known as the city of Shrimps, because 75% of all shrimps exported from Bangladesh are cultivated in the Khulna zone. In addition to this, a major portion of the Golden Fiber (Jute) is exported through Khulna Zone. Khulna is famous for its fish and seafood industries. Lobster, Prawn, Catfish, Shrimp and Crab are now being exported abroad from Khulna.

4.2 Climatic Condition

Khulna city experiences Indian Ocean monsoon climatic conditions. The area experiences four meteorological seasons; pre-monsoon (March to may), monsoon (June to September), post-monsoon (October to November) and dry (December to February). Average annual rainfall is in the range of 1,700 to 2,200 mm. about 70% rainfall occurs during the period from June to September. Mean monthly rainfall during the same period is between 300 to 450 mm. maximum daily rainfall is about 200 mm but it is increasing. In 2004, the maximum daily rainfall was 364 mm. average temperature range is between 25ºc to 31ºc. Maximum temperature may rise up to 40ºc and may go down to 6ºc. Average humidity remains at 80% to 90%.

4.3 Topography

The city area, the central part of the greater Khulna is occupied by the southern half of the Sundarban tract. The rest of the area is covered by the floodplain of the jamuna, paDMA and Meghna Rivers. A remarkable difference in the ground elevation can be observed throughout the Khulna city, which is reflected in distinct landforms: high lands, low lands, and abandoned channels and depressions. Ground elevation of the city varies from 0.5 m to 12 m (pwd). About 60% and 70% of the city area includes low lands, abandoned channels and depressions and the elevation of these areas vary from 0.5 to 5 m (pwd). (Fig 4.1)

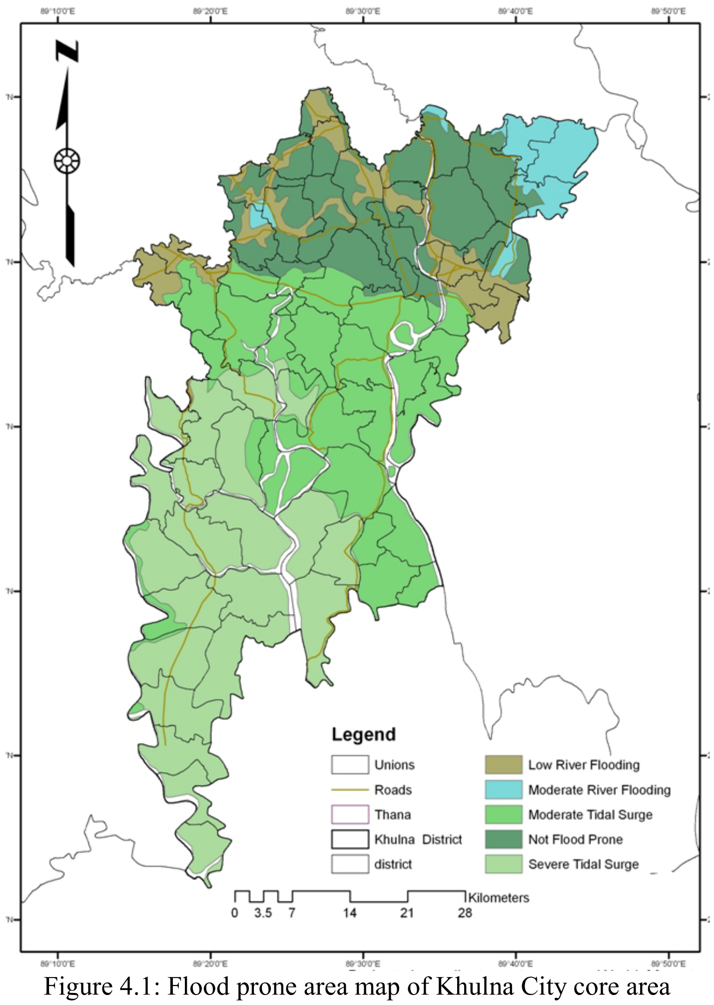

Figure 4.1: Flood prone area map of Khulna City core area

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2 With the sea level rise Mongla Port may have more operations and therefore more activities. Even 1 meter rise will make many coastal river ports more active and navigation will increase further. Creeks, canals and small rivers, which are now not under operations likely to come under operation with sea level rise.

4.4 Meteroligicaal Status

Khulna is the third largest city in Bangaldesh. It is located on the banks of the Rupsha and Bhairab rivers in Khulna District. It is the divisional headquarters of Khulna Division and a major industrial and commercial center. According to the Calcutta Gazettee, Khulna was declared as municipality on September 8, 1884 by the then Governor (http://www.khulnacity.org/). It has a seaport named Mongla at its outskirts, 38 km from Khulna City. The population of the city, under the jurisdiction of the City Corporation, was 855,650 in 2007. The wider Statistical Metropolitan Area had at the same time an estimated population of 1,388,425[3]. The annual average temperature for Khulna is 35.5 °C with a record low of 12.5 °C (Wikepdia).

4.5 Water sources

Source of drinking water: Data (Table 4.1) on source of drinking water show that Khulna have highest number of tap water use while again deep tubewell use also very high in Khulna. In case of other towns dependency on other source is much higher than Khulna. Interestingly there is no dependency on open water use which means some kind of arranged water use is practiced. In 2030 and 2050 dependency on tap water will increase further which means more use of tap water will be there. Shallow tubewell use in Bagerhat is less as there is more salinity in upper level of groundwater.

Source of water for domestic use: Again source of domestic water use (Table 4.1) is different in Khulna from Bagerhat and Sathkhira. Khulna has more dependency mainly on deep tubewell and tap water while other two towns have different scenario. Reasons for having different scenario in those towns are mainly because of low DTW than Khulna. In 2030 and 2050 there will be more dependency on tap water like Dhaka.

Table 4.1: Water sources in Khulna City

Source of drinking water	Khulna District	Domestic Water Source	Khulna District
Tap water	10.00%	Tap water inside house	15.80%
Tap water in front of the house	0.30%	Tap water in front of the house	1.30%
Govt. tap pipe/ road side tap	3.00%	Govt. tap or tap at road side	4.70%
Shallow tubewell (<500 ft)	13.70%	Shallow tubewell (< 500 ft)	12.20%
Deep tubewell (500+ ft)	72.80%	Deep tubewell (<500+ ft)	63.30%
Pond/ irrigation canal	0.20%	Dug well	0.00%
Bottled water	0.01%	Surface water (pond, canal etc.)	2.70%
Total	100.00%	Total	100.00%

4.6 Existing Water Supply System

Residents of Khulna city are suffering from serious water crisis as Khulna Water and Sewerage Authority (Wasa) is currently supplying only nine crore litres of water against the daily demand of 24 crore litres for around 15 lakh people of Khulna city.About half of the remaining 15 crore litre of water is being supplied through 3748 tube-wells installed by Khulna City Corporation (KCC) and 22,701 other private owned tube-wells.And so, a good number of city dwellers have to depend on other sources including ponds for water. Khulna Wasa has 73 large 'production' tube-wells which are used to supply nine crore litres of water through pipelines for the subscribers in Khulna city, officials said. Khulna Wasa has undertaken a Tk 2,500 crore 'Khulna Water Supply Project' that has already got approval from the Executive Committee of National Economic Council for implementation.The water brought from the Madhumati will be purified at the water treatment plant at Shamantashena and supplied through pipelines to the subscribers of Khulna city.The proposed water supply project includes installation of 700 kilometres of pipeline.Works for acquiring land under this project are going on for construction of a water treatment plant.At least 75,000 subscribers will get benefit of water supply with implementation of this project which is expected to be completed by 2017. After completion, it will be possible to supply more five crore litres of water a day in Khulna city.

[3]Md. Rejaur Rahman, Urban Spatial Growth Analysis of Khulna City, Coordinator & Urban Planner Asian Disaster Preparedness Center (ADPC), internet version.

4.7 Prolems came out from field level discussions

Increased water level in the rive causes lot of problems. It causes flooding and water backflow in the residential areas. In the rainy season, when incessant rain occurs, the run-off does not flow to the river which entails water congestion. It is a big problem in some parts of the Khulna city. Duration of this water congestion remains 2-3 days. Different types of problems are emanating from water congestion which includes affect to work, movement, sanitation, business and even schooling which get suspended. Also poor people take shelters at different places and different types of water borne diseases occur in that monsoon time.

Water logging: Actions taken or adaptation made for water logging include transfer the valuable assets to other areas (6.2%), followed by sent the family members out of disaster area (5.0%), raise homestead level /plinth level (3.5%), or even wait for relief, etc. in Khulna. In terms of degree of adaptation Khulna and Bagerhat is the same while Sathkhira is different. One reason for having this trend is that the intensity of economic activities in Khulna and Bagerhat is also higher. In 2030 and 2050 water management crisis likely to be different with climate change and adaptation also likely to be different with further investment during that period.

Heard about climate change: People in general heard about climate change where more heard about CC in Bagerhat and Sathkhira. About the information that they got highest in per cent include hot temperature (98.1%), very cold (99.5%), water logging (6.4%) in Sathkhira, change in rainfall (61.2%) in Khulna, frequent cyclone (12.8%), and frequent flood (8.2%) in Bagerhat. All those indicate to the fact that the people in general are aware of CC. But it does not mean people are equally prepared to address the challenges of CC, rather they may feel more vulnerable and therefore, victim of the CC. In 2030 and 2050 it is expected that people will not only become more aware of CC rather they will directly experience even more adversity of CC.

Crisis of sweet water due to climate change: Crisis of sweet water due to climate change is expected to be highest (76.3%) in Khulna.The process of meeting those crisis or adaption measures suggested by the respondents include installation of deep tube well (60.8%), manage by tap water supply (8.0%), drinking boil water (6.1%), and collect rain water (3.8%). Out of all that highest percent of support to the DTW is mainly because they are currently used to DTW only and they see that as safe mode. Most of them (87.7%) mentioned that they could face it. Almost all (97.3%) mentioned that they need some assistance from the government, NGO (37.5%), and local people (21.8%).

Analysis:

Water cycle:

Loss (Intangible)	Damage (Tangible)
Standing water increased heat conduction into the soil, leading to higher soil temperature. The soil loses its structure and texture.	Water quality damaged by salinity intrusion.

Health aspect:

Loss	Damage
Epidemic diseases losses lots of human life.	Several flood borne diseases like Dengue, Leptospirosis (animal), etc breaks out severely.

Resource and quality Management:

Loss	Damage
• Losses of both renewable and nonrenewable resources. • Losses of human life causes human resource life and quality of life.	• Damage of water quality, fish culture and agriculture. • Damage of capital like boat, farm land etc for village economy.

Thousands of households get affected at different hotspots of Khulna city. Hotspots which get seriously affected by water supply problems include Diyana School area under ward no. 4, Thana Doulatpur; Khalishpur, ward no.

08, Thana Khalishpur; water congestion affects Khulna Medical Collage, Ward: 17, Thana Sonadanga; Sonadunga Bus Stand, Ward no 18, Thana Sonadanga; Duckbanglo, Ward: 21, Thana Sadar; Feerighat, Rupsha, Ward no 21, Thana Sadar; flood affects Sangita More, Ward: 25, Thana Sadar; and Hotel Royel More, Ward: 25, Thana Sadar; drainage problem affects Shib Bari More, Ward 25, Thana Sadar; and submerged with water with heavy rainfall or drainage problem affects Forizi Para, Ward: 28, Thana Sadar. The inhabitants suffer in rainy season and it becomes more severe when water congestion remains for 1-2 days to about 1 to 2 month. Almost all people suffer in rainy season. City dwellers face serious problem during the disaster like diarrhoea, cholera, dysentery, typhoid, etc. During the problem they do not move to other places as there is no cyclone shelter.

5. CAUSES OF LACK OF SAFE DRINKING WATER IN KHULNA

5.1.1 Arsenic Contamination

Underground water of south-eastern region contains arsenic. A study carried out by a local NGO indicated that 79% of the tested tube-wells of the area contained arsenic beyond the acceptable limit (http://pravdabangladesh.wordpress.com/access-to-safe-drinking-water/). Most of the shallow tube-wells in the region draw either brackish water or arsenic- contaminated water.

5.1.2 Lack of aquifer

Ground water occurs in permeable geological formations known as aquifers. For extraction of groundwater medium clean sand is suitable. This sand has considerable porosity and permeability and can store a huge amount of water. Fine sand also can store a considerable amount of water. However, as the position of the area is in the lower part of Ganges delta the sediments of the region have very low permeability and are not able to store water. As a result, the region lacks aquifer that fresh groundwater can be extracted from.

5.1.3 Cultivation of brackish water shrimp

In the southwest region shrimp cultivation is underway in almost all the wetlands. In most of the cases, salt water from the river is brought into the wetland for shrimp cultivation, which is increasing the salinity of the adjacent fresh water ponds and shallow aquifer through seepage (WorldWide, DanChurchAid, MuslimAid, Relief, Oxfam-GB, & Children-UK, 25-31 October 2009). Thus there is great scarcity of drinking water in areas where shrimp is cultivated and that covers greater portion of southern districts of this region. This shortage of drinking water affects women the most, as it is their responsibility to collect drinking and cooking water for the household. They have to walk several kilometers to obtain drinking water, wasting much of their time that they could have used in productive employment.

5.1.4 Reduction in upstream flow

In the past the southwest coastal region was rich in fresh water as the Ganges had flowed through it. However, the scenario changed following two disastrous events: the change of the course of the river Ganges due to Ganga water distribution Treaty, commonly known as Farakka Treaty due to which only 27500 thousand cusec water becomes available for Bangladesh during the dry period with the remaining amount being diverted by India) and the closing of the face of the origin of the river Matha Vanga ((http://pravdabangladesh.wordpress.com/access-to-safe-drinking-water/)). This had a serious implication for safe drinking water available from ground water sources. The reduction of upstream flow deteriorates the recharge rate of the ground water table, reduced fresh water bodies and results in over extraction of groundwater for irrigation and use of water from fresh water ponds.

5.1.5 Excessive use of underground water in an unplanned way

Since the 1980s vast land in the southwest coastal region, except the slight saline wetland, has been brought under irrigation for cultivation of Boro rice through extraction of underground water in the dry season (Hafizi N, 2011). The lack of surface water for irrigation during dry season has compelled the farmers to exploit underground water extensively resulting in a lowering of underground water table beyond the suction limits of shallow tube-well, making millions of shallow tube-wells dysfunctional. This over-extraction of groundwater is one of the possible reasons for the contamination of shallow aquifer by arsenic.

5.1.6 Natural disasters

Due to geographical disadvantage, this southeastern region of Bangladesh regularly experiences natural disasters (e.g. water logging, cyclones, tidal surges, floods, river erosion, etc) which are responsible for the destruction of drinking water sources. In addition, effects of climate change have caused hazards in this region to occur more frequently than before and with greater intensity. During cyclone Sidr in late 2007 the majority of drinking water sources became dysfunctional. Under the Sidr rehabilitation programs water supply and sanitation facilities were restored by various government and non government agencies. However, the majority were again damaged by the recent cyclone Aila (Dasgupta, et al., 2011).

6. FUTURE PLAN FOR WATER RESOURCES ADAPTATION IN KHULNA ZONE

Human health: the health care system in Bangladesh falls under the control of the ministry of health and family planning. The government is responsible for building health facilities in urban and rural areas. During dangerous months like monsoon season and other natural disasters, the Bangladesh health system isn't capable of managing the number of victims.as the most of the doctors are willing to practice in the urban area.
The adaptation options have been categorized into two types:

- Core options: major measures that directly and largely solve the main issues, e.g. location of surface water intake, impounding reservoir size, increase of drainage capacity, etc.
- Add-on options: useful additional measures but which by themselves will not be able to solve the main issues fully, e.g. rainfall harvesting, demand oriented measures (such as progressive pricing).

Add-on Options

Several add-on options have also been identified and the proposed list is provided in Table 7.1. These add-on adaptation options are unlikely to address the identified climate change impacts significantly when implemented by themselves. A qualitative assessment of these add-on options will be provided by the socio-economic group, after the stakeholders have provided feedback on the proposed list.

Table 7.1: Proposed Water Supply Add-on Adaptation Options

Add-on Option Description	Remarks
Reduce leakages	• Requires good operation and maintenance of distribution system.
Water demand Management	• Includes progressive water pricing • Includes awareness raising campaigns • There may be public resistance to water price increases
Dual reticulation (2 pipe system)	• Includes progressive water pricing • Includes awareness raising campaigns

Actions taken for excessive salinity in water: Many measures are taken due to excessive salinity in water. The trend is evident in case of khulna where people take measures widely. They used to carry water from other areas (41.0%) followed by buy water from others (37.5%), reduce water use in washing (29.5%), etc. In the case of Khulna city as there is wide use of deep tubwell, there is less use of saline water. In 2030 and 2050 more action would require from water suppliers' side.The criteria for water supply options will be slightly different. For example, damage costs are not relevant for assessing the water supply options.

Economic Criteria: The economic criteria mainly involve the costs for each of the core adaptation options. The usual costs (investment and O&M) will be estimated.

Social Criteria: Quantitative social criteria include the number of people affected (positively and negatively) by the adaptation option, diarrhoea incidence rate, number of jobs created during construction phase of the option and the water price (for supply options). Two qualitative criteria are included: public acceptance of the adaptation option and an assessment of the overall health impact of the option.

Environmental Criteria: Two quantitative criteria include the number of species affected and the number of sensitive sites affected by the adaptation option. It will not be possible to assess this in detail but an approximate number based on secondary information sources will be used. Two qualitative criteria include the degree of pollution expected during the construction and operation phases of the core adaptation options.

Other Criteria: A couple of additional criteria that will be used include: robustness and flexibility. Robustness expresses how effective the strategy is also under different future developments – in other words: if the strategy is not sensitive to changes in the scenario variables. For example, for urban water supply, the DTW option can be considered a robust solution if very deep tubewells are used. This option would then not be sensitive to changes in the scenario variables.

7. RECOMMENDATIONS

The expected climate change impact on water supply options is increased river salinity. Therefore, the number of days that river water is not suitable may increase compared to the base (no climate change) scenario. Thus one core option is to increase the impounding reservoir size so that it can provide alternative supply for a longer period. Another core option is to move the intake point further upstream where the salinity level in the climate change scenario is similar to that in the base scenario at the original intake point. These two types of core adaptation options have been selected based on practicality and cost-effectiveness. Another practical core option included is mixing of groundwater with river water at Dakatia Beel reservoir site. This is due to the close proximity of a well field to the proposed impounding reservoir.

During the problem their livelihood gets affected but they have little to do. To overcome it some measures may be taken, such as, needs to improve the pure water supply and to set up safe sources even in the city areas. They believe that all problems are curable, only needs genuine will to do it. To overcome this problem some other measures may be taken like to ensure pure drinking water, to construct embankment, construct disaster rehabilitation center and create awareness about climate change.If two pipe-lines are installed – one for drinking water and other for domestic use- both are necessary. In dry season when water layer goes down and the tube-well becomes useless then it may be necessary for drinking and domestic uses. If water is available all the time then they will not mind to pay for it. Water means life and when they face water crisis sometimes they buy bottled water for drinking. They don't use any other means to get water.

8. CONCLUSIONS

Several households get affected for pure water sorce with the rise in water level in Bairab river.All people use deep tubewell water. In deep tubewell water some iron was observed. They have to depend on nature as government agencies do not do anything needful. Sometimes they go to the leader of the area who just gives hope to them. They see health hazard caused by drainage problem such as diarrohea, dysentery, cholera, and mosquito bites. They have to sit idle during that time as they have nothing to do. Due to that they reduce consumption of food also.Some measures may be taken such as rehabilitation of drainage system, control over mosquito, more tree plantation and construct embankment so that water cannot enter inside the city. Ward councellors managed to open the sluice gates and they clear the drain near to their houses. People face various problems during disasters period like roads get destroyed, houses get affected, and they face health hazard. During the problem water enters into their houses and they temporarily migrate to other safe places. Rich people suffer less where poor, women and child suffer more.So we need more concern for them all time.

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REFERENCES

Engr.Md.Shahadat Hossain (2013), Alternate water supply options for disaster prone areas in bangladesh: a case study on Hatiya pourashava, A Dissertation for the Degree of Post Graduate Diploma on Disaster Management By, IDMVS , Dhaka University.

Bangladesh WASH, K. B. (09-10 December 2009). Learning and Knowledge Sharing Workshop on Response to Cyclone Aila Khulna. Water Aid.

Board, T. SWOT Analysis and Sustainability Study for A Service Sector-A Case Study of TWAD Board. TWAD Board.

Haider, D. (14 July 2011). Solution Exchange for the Climate and Disaster Risk Reduction Community Consolidated ReplyQuery: Safe Water for the South West – Experiences.

Sharmin, F. (20 April 2010). Sustainable Alternative Water Supply Technologies in Arsenic Prone Areas of Bangladesh.

Zaman, M.(2011), Climate Change, Cross border Migration and Regional Conflicts: Risks and Strategies for Bangladesh; Bangladesh Calling for Climate Justice.
John Morrison (2004), Managing leakage by District Metered Areas: a practical approach.
LGD, MoLGRD&C, Drainage Policy for Dhaka City, June 2005.
Rahman, Md. Rejaur, Urban Spatial Growth Analysis of Khulna City, Coordinator & Urban Planner Asian Disaster Preparedness Center (ADPC), internet version.
http://en.wikipedia.org/wiki/Khulna
http://www.khulnacity.org/

Proceedings of the 2nd International Conference on Civil Engineering for Sustainable Development (ICCESD-2014), 14~16 February 2014, KUET, Khulna, Bangladesh, ISBN: 978-984-33-6373-2 (CD-ROM)

EFFICACY OF TUBULAR SOLAR STILL (TSS) IN PRODUCING DRINKING WATER FOR SOUTH-WESTERN PART OF BANGLADESH

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ABSTRACT

In this paper, perfomance stduy of six PSF's from Paikgacha, a typical arsenic contaminated Upazilla in the coastal belt of the south-western region of Bangladesh and efficacy of low cost TSS have been conducted throughout the project duration (May 2012 to April 2013) to justify whether a TSS could be an appropriate solution for such regions to meet the fresh water demand for drinking purpose or not. The quality of the raw water from the PSF is found extremely poor. Turbidity, dissolve solids and total solids values of both raw and treated water samples are found within the allowable limits. In most cases, values of suspended solids exceed the allowable limits, although there is some improvement of suspended solid values observed in treated water. The total coliform, E.coli and BOD_5 values represents that the treated water of the study area is unsuitable for drinking as all the samples exceeds the allowable limits. The performance of existing PSF are found not satisfactory. The maximum and average daily production from the TSS are found as 4.26 and 2.10 lit/m^2-day (0.98 and 0.48 lit/day), respectivly. The production cost of water from the TSS is estimated as 0.27 Tk./lit. The construction, operation and maintenances of TSS are simple and environment friendly and also the construction and the water prduction cost of TSS are found very low. It is concluded that the TSS could be an appropriate solution for such regions to meet the fresh water demand for the drinking purpose.

Keywords: *Arsenic, Pond Sand Filter (PSF), Solar Radiation, Salinity, Tubular Solar Still (TSS)*

1. INTRODUCTION

In recent years, peoples living in coastal, remote or arsenic contaminated areas of Bangladesh have been received global attention due to scarcity of fresh water for their living purpose. Contaminated wells exceeding the Bangladesh standard of 0.05 mg/l have been identified in 41 of the country's 64 districts. It has been estimated that a population of 25 to 36 million are exposed to arsenic contamination and related health hazard risks. It is estimated that approximately one-third of the world's population use groundwater for drinking (Nickson et al., 2005). However, in the coastal regions in any country, groundwater quality patterns are complex because of the input from many different water sources (Ramkumar and Anita, 2010). These include precipitation, seawater, ascending deep groundwater, and anthropogenic sources such as wastewater or irrigation return flow. The main sources of water in Bangladesh are surface waters in rivers, reservoirs, lakes, canals and ponds, and ground water in deep and shallow aquifers. There are certain areas in the coastal belt of Bangladesh where tube wells are not successful, because ground water is mostly saline upto depths of 700-1000 ft. (DPHE-UNICEF, 1989) and suitable freshwater aquifers are not available. In addition to this, arsenic contamination of ground water in Bangladesh has been recognized as major problem since 1993. The concentration of arsenic in excess of allowable limits has been found in shallow tubewells in many parts of Bangladesh. Thousands of people have already been identified to be affected by arsenic poisoning, in addition to the millions potentially under threat from drinking contaminated water (Ahmed and Rahman, 2007). Provision of arsenic free water is urgently needed for immediate protection of health and well being of the people living in arsenic affected areas. However, to ensure the safe water for drinking and all other household usage, the concerned persons and organizations (Department of Public Health Engineering (DPHE)-Danida, UNICEF, WHO, NGO Forum etc.) are searching for appropriate alternative technology for the coastal belt. One of the alternative sources of potable water for this area appears to be the development of the appropriate desalination technology. Pond Sand Filter (PSF)s were first introduced by DPHE-UNICEF jointly to overcome the problem since 1984 on a pilot basis. The NGO Forum has also been implementing PSF and rainwater harvesting system since 1997 in the coastal area of Bangladesh. The people in many places of southern fringes of Khulna, Noakhali, Bagerhat, Satkhira, Barguna, Perojpur, and Patuakhali districts are mainly depending on PSF as community base system and rainwater harvesting as household base system to meet their fresh water demand throughout the year.

Most desalination techniques consume a large amount of energy. Moreover, many remote towns and communities rely on costly and often limited supplies of diesel fuel for their energy needs. These and other forms of fossil fuels are sometimes heavily subsidized by government to meet community service obligations (Water Corporation, 2000). Therefore finding methods of using renewable energy to power the desalination process is desirable. Solar distillation is the simplest desalination technique, compared with other types, e.g., multiple-effect distillation, multi-stage flash, reverse osmosis, electro-dialysis and biological treatment due to no need of fossil fuel or electricity. A basin-type solar still is the most popular method of solar distillation, but main drawbacks of the basin type are not easy of construction and the difficulty in rapid and easy removal of basin accumulated salt. Therefore, I designed a new type of low cost solar distillation unit, Tubular Solar Still (TSS), to overcome such difficulties in the maintenance and management. In this study, Paikgacha a typical arsenic contaminated Upazilla in the coastal belt of the south-western region of Bangladesh has been investigated to study the present situation of fresh water condition and a low cost TSS was designed, constructed and field experiments have been carried to justify whether a TSS could be an appropriate solution for such regions to meet the fresh water demand for drinking purpose or not.

2. METHODOLOGY

For better understanding the present scenario of existing drinking water facilities in a typical arsenic contaminated coastal region, Upazilla Paikgacha has been selected for the present study. To identify the technical and social problems of existing fresh water facilities, a detail field investigation has been carried out. Raw water and filtered water samples have been collected from selected six PSFs and water quality parameters such as total coliform, fecal coliform, turbidity, salinity, pH, TS, TDS, SS, alkalinity and color have been tested. A low cost Tubular Solar Still (TSS) is designed and constructed using locally available materials. Field experiments are conducted using the constructed TSS. Daily distilled water production and are recorded. Collected data was analyzed and correlations are proposed for daily output. Finally, the water production cost is estimated and conclusions are drawn whether a TSS could be an appropriate solution for such regions to meet the fresh water demand for drinking purpose or not.

3. STUDY AREA AND DATA COLLECTION

Upazilla Paikgacha under Khulna district has been selected for the present study. It is one of the typical arsenic contaminated Upazilla in the south-west coastal belt of Bangladesh. It is located about 62 kilometers west of the district headquarters of Khulna and about 350 kilometers south from the capital Dhaka. The upazilla is divided by the river, Shipsa and the household of this area are mainly living on both side of the river. The land of this area is defined as medium high land, and the soil quality is alluvial, loamy and sandy. A detail field investigation has been carried out on the existing facilities of fresh water. The available sources of water are ground water and surface water. The ground water contains excessive arsenic and salinity which exceeds the allowable limits. So the ultimate source is the surface water namely pond water. The pond water is being used by filtration called PSF.

Six PSF's shown in Table 1 under Paikgacha upazilla have been selected on the basis of preliminary investigation to monitor the performance of the PSF's. It is to be mentioned here that all the PSFs were constructed by DPHE-Unicef. A map showing the study area and PSF locations are given in Figure 1. Raw and treated water samples from the selected PSF's have been collected once in a month from May 2012 to April 2013. The samples from the selected PSF were collected on May 12, June 9, July 14, August 11, September 8, October 13, November 14 and December 8 of 2012 and January 12, February 9, March 9 and April 13 of 2013. Then the collected samples were tested in Environmental Engineering laboratory of Khulna University of Engineering & Technology (KUET), Khulna for analyzing the water quality parameters. Figure 2 shows some photograhs of sample collection from the selected locations.

4. DESIGN, CONSTRUCTION AND FIELD EXPERIMENT OF TSS

The TSS has been designed and constructed using the locally available materials. It is composed of tubular cover and a black rectangular trough in it for storing brackish or saline water. The tubular cover is 110cm long, 30cm in diameter and made of 2.75mm thick GI wire frame covered with transparent polythene paper. The trough is 105cm long, 28cm wide, 7cm deep and made of 2.5cm thick ferro-cement material. The distilled water is collected in a PET bottle and the bottle is kept in an insulation box. A pair of Ferro-cement L-shape frame is used to support the TSS. The schematic diagram of the TSS is shown in the Figure 3.

Table 1: List of selected PSF's for the study area.

Serial No.	Name of the Village	Locations	
		Latitude	Longitude
1	Sheikhpara	22°38'59"N	89°18'32"E
2	Maloth	22°39'08"N	89°18'40"E
3	Gopalpur1	22°36'11"N	89°18'32"E
4	Gopalpur2	22°36'12"N	89°18'12"E
5	Paikgacha Madrasa 1 (old)	22°35'00"N	89°19'46"E
6	Paikgacha Madrasa 2 (new)	22°35'00"N	89°19'46"E

Figure 1: Existing locations of pond sand filter in Paikgacha Upazilla.

a) Sheikhpara

b) Gopalpur

Figure 2: Photographs showing the collection of samples from selected PSF's.

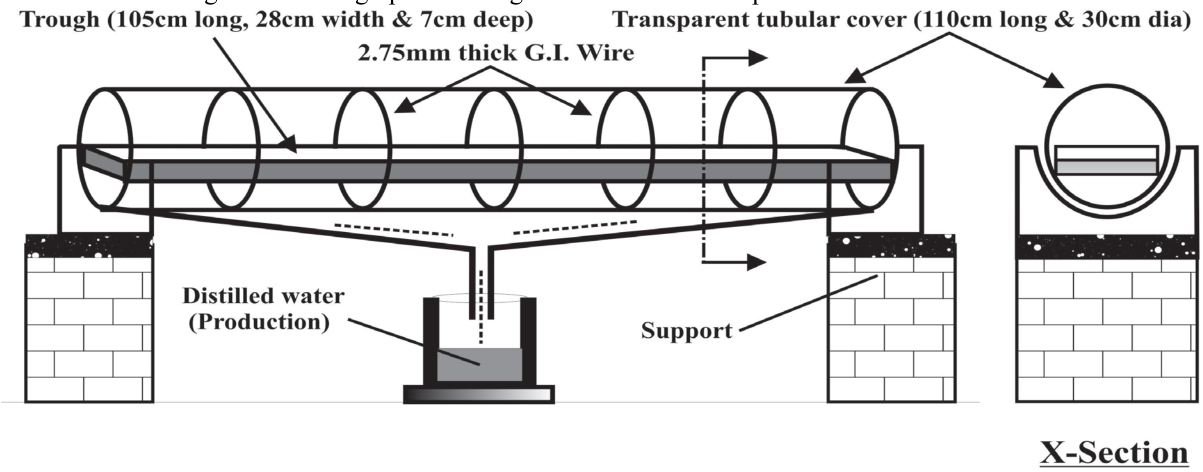


Figure 3: Schematic diagram of the TSS.

Two TSS's were constructed to conduct the field experiment. The field experiments have been carried out on the roof top of the Civil Engineering building of Khulna University of Engineering and Technology (KUET) throughout the project duration (May 2012 to April 2013). Figure 4 shows the photograph of the field experiment. Supports are provided at both ends of the TSS so that free circulation of air occurs beneath the still. A PET bottle was used for the collection of distilled water output and was put in a wooden box. The daily output from the still is collected approximately two hours after sunset. Hourly output for some typical days are also recorded. The bottom side of the tubular cover is made sloping downward at middle so that distilled water output can be collected easily in the PET bottle. One end of the TSS was kept fixed and other end could open to clean the trough or to remove the accumulated brine and to feed the saline water in the trough. To check the performance of the TSS, solar radiation flux and ambient air temperature were also measured at one minute interval using a data logger. A pyranometer and thermocouples were used to measure the solar radiation flux and temperatures, respectively.

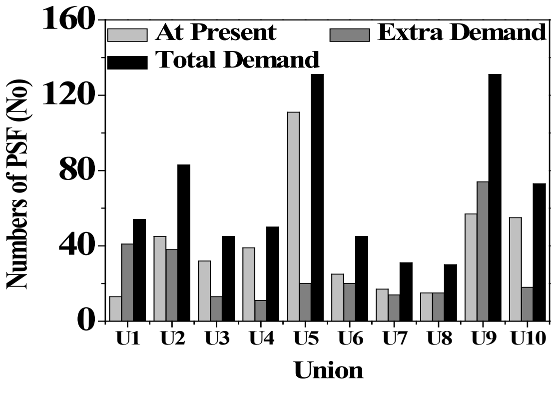
Figure 4: Photograph of the field experiment

5. RESULTS AND DISCUSSION

Paikgacha is a typical arsenic contaminated Upazilla in the south-west coastal belt of Bangladesh. The available sources of water are ground water and surface water. The ground water contains excessive arsenic and salinity, which exceeds the allowable limits. So the ultimate source is the surface water namely pond water. The pond water is being used by filtration called PSF. Many pond sand filters is being chocked up during rainy season and polluted through the natural pollutants. It is situated in a flood affected zone.bAlmost every year flood is occurred and flowing through the alluvial plain. In recent two natural storms namely Sidor and Aila had occurred and inundate the whole area. The resulting effect of Aila over stress fresh water supply of that area. Also this flood water mixed with various waste and increase coliform and salinity. In this situation the pond water is totally unhygienic for drinking. Use of such contaminated water is the cause of various diseases like diarrhea, dysentery, cholera among rural people, which results a chronic impact over rural health and economy of Bangladesh. Recently rain water harvesting is becoming an alternative source for drinking water. The area of Paikgacha is 357.76 km^2, total number of household is 55,664 and overall population is 2,95,732. Details of Paikgacha upazilla are given in Table 2. Figure 5 shows the comparison of PSF demand for all Unions of Paikgacha Upazilla.

Table 2: Details of Paikgacha

Union	Area (km²)	No. of Family	Population	Details of PSF		
				At Present	Extra Demand	Total Demand
Horidali	18.75	5917	30886	13	41	54
Kopilmoni	37.85	6995	38192	45	38	83
Lata	43.47	2739	14379	32	13	45
Deluty	43.58	3896	19805	39	11	50
Soladana	44.47	5179	26468	111	20	131
Laskar	41.92	3961	25050	25	20	45
Gadaipur	18.79	5511	28215	17	14	31
Raruli	21.56	6444	35452	15	15	30
Chandkhali	40.37	10147	51954	57	74	131
Goraikhali	47.00	4875	25341	55	18	73
Total	357.76	55664	295732	409	264	673
In addition 380 household rain water harvesting system also exists.						



U1= Horidali, **U2**= Kopilmoni, **U3**= Lata, **U4**= Deluty, **U5**= Soladana, **U6**= Laskar, **U7**= Gadaipur, **U8**= Raruli, **U9**= Chandkhali, **U10**= Goraikhali

Figure 5: Comparison of PSF demand for different Union of upazilla Paikgacha.

5.1 Water Quality of Selected PSF's

The collected raw and treated water samples from the selected PSF's were tested in the Environmental Engineering laboratory of KUET, Khulna, Bangladesh for water quality parameters such as total coliform (TC), E.coli (EC), turbidity, salinity, pH, total solids (TS), dissolved solids (DS), suspended solids (SS), BOD_5 and color. DS, EC and p^H were determined by TDS meter, Conductivity meter and p^H meter, respectively. Color and turbidity were measured by Spectrophotometer and Hellige turbidity meter, respectively. Salinity as chloride was determined by titration method. All the measured values of water quality parameters are also compared with the WHO and BDS for evaluating the vulnerability. The measured values of different physical and chemical water quality parameters of the collected water samples are summarized in

Table 3. Findings of the study have been assessed according to WHO drinking water quality guidelines and Bangladesh Standards (BDS) as well.

Figure 6 shows the comparison of water quality parameters for both Raw and Treated Water samples from the selected PSF. The p^H values of almost all the water samples were found within the allowable limit. The pH values were varied from 6.72 to 8.59 and 6.81 to 8.59 for the raw and treated water, respectively. The color values varied from 8 to 495 and 0 to 155 (Pt-Co) for the raw and treated water, respectively. Color in water is primarily occurred due to the presence of colored organic substances (primarily humic substances), metals such as Fe, Mn or highly industrial wastes. Hence, it clearly indicates that the removal effeiciency of color is higher in treated water.

The turbidity values varied from 1.11 to 37.50 NTU for the raw waters, wheres it was found within the allowable limit (10 NTU) for the treated water (0 to 8.33 NTU). Hence, high removal efficiency of turbidy is found in PSF's. The salinity values are varied from 100 to 1290 and 30 to 1045 mg/l for for the raw and treated water, respectively. Hence less improvement for salinity has been found in treated water.

Total solids and dissolved solids are found almost within the allowable limits, whereas suspended solids are found higher for both raw and treated water. It is also seen that less or very low improvement for suspended solids have been occured for the treated water. The values of BOD_5 are varied from 0.11 to 4.46 and 0.10 to 1.68 mg/l for the raw and treated water, respectively. Hence, significant improvement of BOD_5 is observed in treated water.

The values of coliforms are found always exceeding the allowable limit (0 No./100 ml) for both raw and treated. Theoretically, it is said that PSF has high removal efficiency of Bacteria. But in some cases, coliforms are found higher in treated water than raw water. This happens due to lack of technical knowledge and poor maintenance and management of PSF. Therefor, no significant improvement of coliforms removal is observed in treated water.

—□— Sheikhpara —○— Maloth —△— Gopalpur-1 —▽— Gopalpur-2 —◇— Madrasa-1 —◁— Madrasa-2

Table 3: Variation of results of water quality parameters for both raw and treated water samples collected from the selected PSF from April 2012 to May 2013.

Sl. No.	Sample Type	p^H	Color (Pt-Co)	Turbidity (NTU)	Parameters (mg/l)					Coliform (No./100ml)	
					Cl-	TS	DS	SS	BOD_5	TC	EC
1	RW	6.72~8.15	37~222	1.31~17.7	150~350	240~520	200~370	30~180	0.52~3.9	84~242	40~200
	TW	7.06~8.16	13~81	1.33~7.60	85~210	290~520	180~340	20~180	0.26~0.85	10~223	5~168
2	RW	7.11~8.10	60~231	1.11~31.7	100~280	250~490	130~350	20~180	0.50~1.33	28~178	8~124
	TW	7.37~8.07	5~48	0.5~5.99	50~220	190~410	150~370	30~70	0.24~0.91	24~211	2~117
3	RW	6.79~7.90	50~105	1.45~19.50	170~1000	440~1260	260~840	40~450	0.50~3.74	46~183	15~93
	TW	6.81~7.79	10~155	0.82~4.30	30~860	420~1060	260~840	10~180	0.24~1.55	22~226	14~118
4	RW	6.93~7.49	8~212	2.17~29.40	350~1290	480~1590	460~1310	20~320	0.77~2.91	17~101	10~73
	TW	7.15~7.79	0~24	0.33~4.21	150~1030	540~1170	450~1060	20~250	0.17~1.68	20~175	3~167
5	RW	7.22~8.59	21~495	2.58~37.50	150~510	380~950	340~920	30~200	1.00~4.46	20~98	10~54
	TW	7.24~8.56	0~102	0.84~7.58	85~290	180~920	170~860	0~80	0.11~1.07	13~194	6~83
6	RW	7.22~8.59	21~495	2.58~37.50	150~510	380~950	340~920	30~200	1.00~4.46	20~98	10~54
	TW	7.25~8.59	0~54	0.92~7.43	88~270	177~840	163~830	10~90	0.10~0.9	11~123	2~12
Allowable Limits											
Standard 1		6.5~8.5	0	10	150~600*	1000	1000	10	0.2	0	0
Standard 2		6.5~8.5	0	5	250	1000	1000	0	--	0	0

Sl. No. 1 = Sheikhpara, 2 = Maloth , 3 = Gopalpur1, 4 = Gopalpur2 , 5 = Paikgacha Madrasa 1, 6 = Paikgacha Madrasa 2
TW = Treated Water, RW = Raw Water, Cl- = Salinity as Chloride ion concentration, TS = Total Solids, DS = Dissolved Solids SS= Suspended Solids, BOD_5 = Biochemical Oxyzen Demand for 5 days duration, TC = Total Coliform, EC = E.Coli
Standard 1: Bangladesh Standard (ECR, 97); Standard 2: WHO Guidelines (1996); *1000 for coastal areas of Bangladesh
Sheikhpara, Maloth , Gopalpur1, Gopalpur2 , Paikgacha Madrasa 1 (old), Paikgacha Madrasa 2 (new)

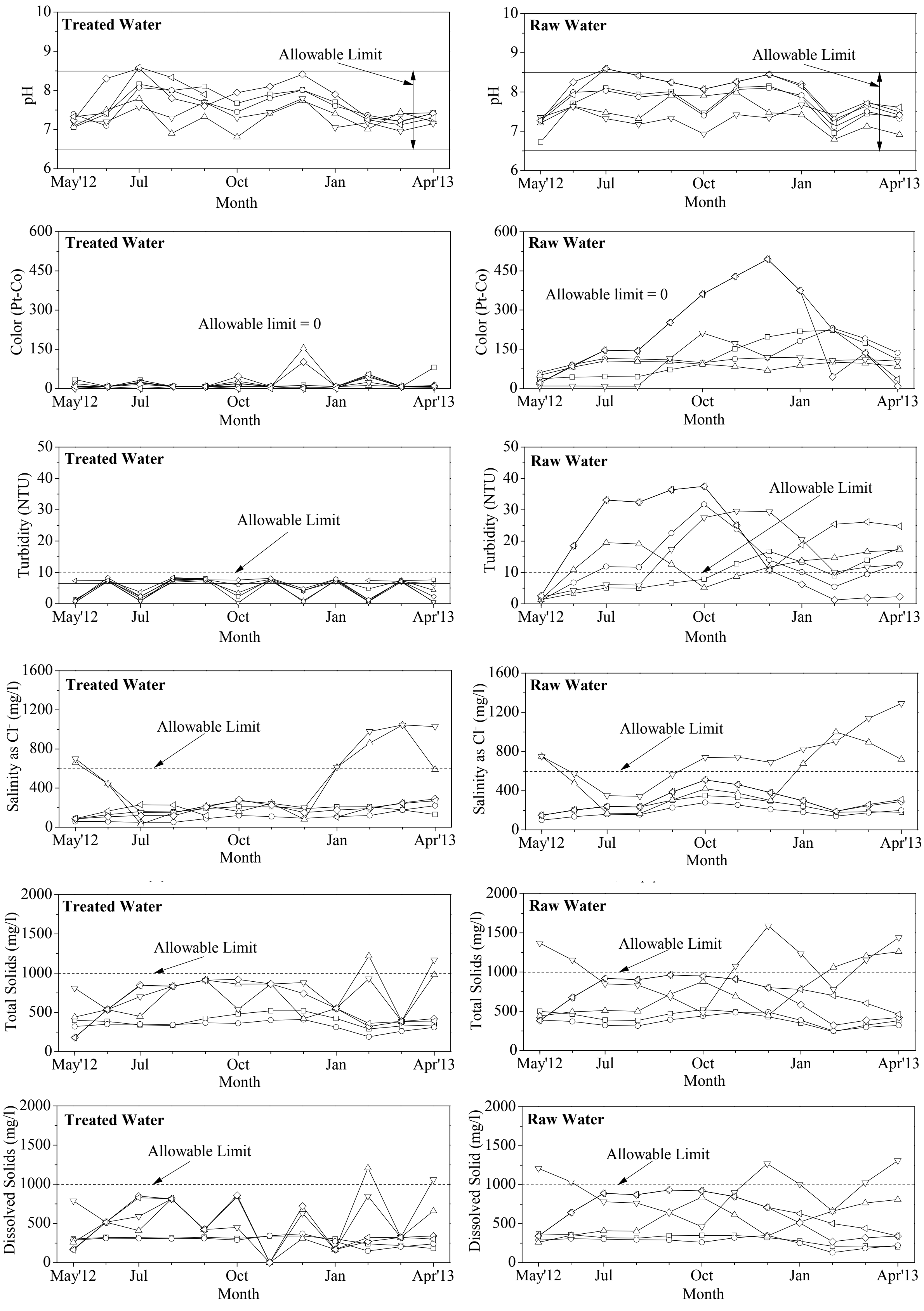


Figure 6: Comparison of water quality parameters for both Raw and Treated Water samples from the selected PSF.

Sheikhpara — Maloth — Gopalpur-1 — Gopalpur-2 — Madrasa-1 — Madrasa-2

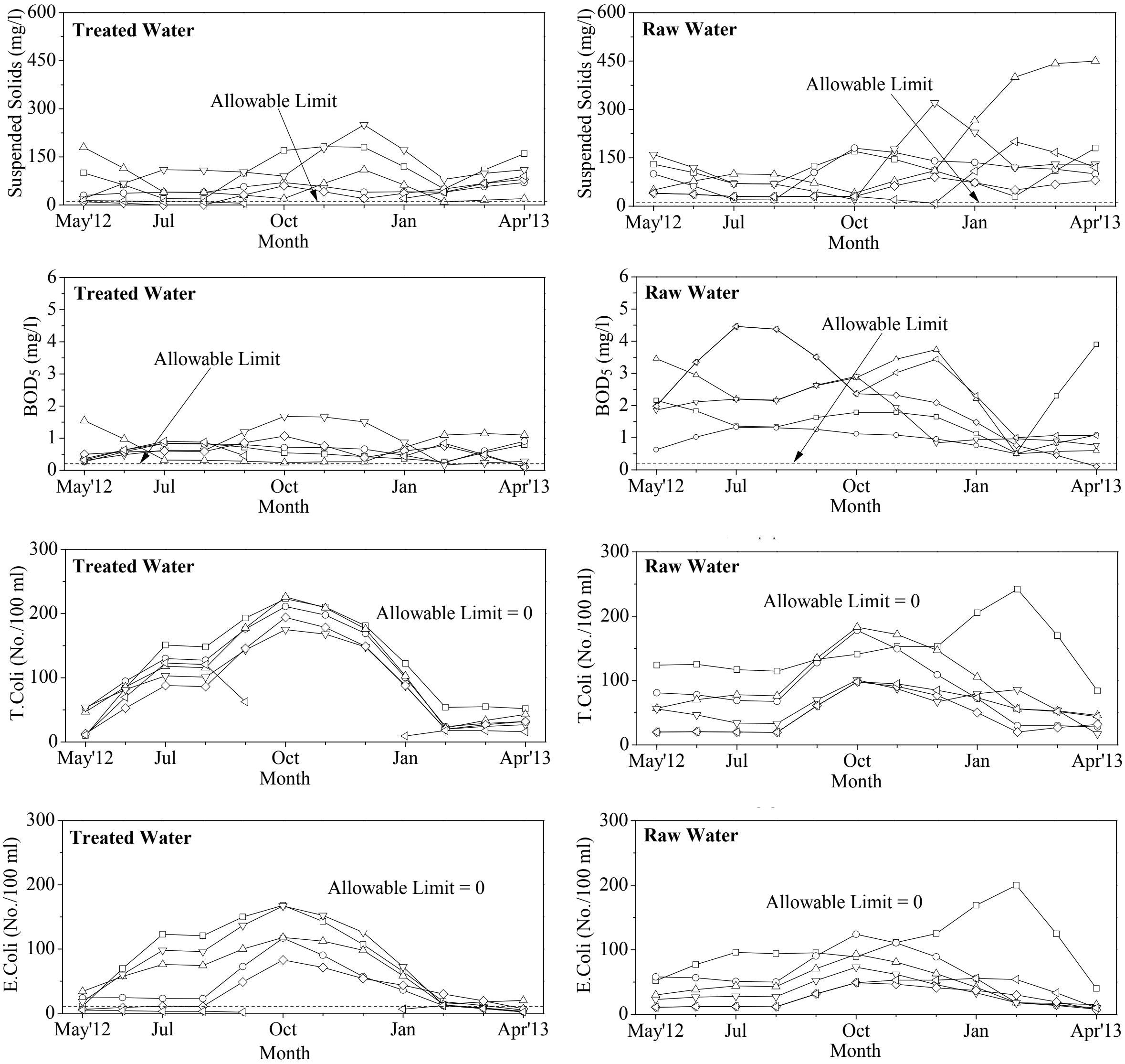


Figure 6 (contd..): Comparison of water quality parameters for both Raw and Treated Water samples from the selected PSF.

5.2 Productivity Analysis of TSS

Two TSS's were constructed to conduct the field experiment. The field experiments have been carried out on the roof top of the Civil Engineering building of Khulna University of Engineering and Technology (KUET) throughout the project duration (May 2012 to April 2013. The daily output from the stills is collected approximately two hours after sunset. Hourly output for some typical days are also recorded to check the hourly production flux. Hourly and dailty distilled water output from the TSS are used to calculate the daily and hourly production rate per unit surface area of the saline water in the trough. Figure 7 shows the observed diurnal variations of hourly production per unit saline water surface area, solar radiation flux and ambient air temperature for the TSS at KUET, Khulna for April 25 of 2012. It is observed from the figure that the solar radiation flux rose rapidly after sunrise (approximately 6:30) and peaked approximately 12:00 after declining gradually. The air temperature also rose gradually in the morning (approximately 7:00) till 13:00, and declined gradually in the afternoon. Whereas the production was recorded from 8:00 in the morning (clearly indicating that there is a distinct time lag between evaporation and production or condensation), increased gradually up to 13:00, and then declined in the afternoon. It was also seen that the slope of the hourly production rate in the morning is steeper than that of the afternoon. The total distillate output for the day is found as 3.76 lit/m^2 (865 ml).

The maximum and average daily productions from the TSS for each months are calculated and tablated in Table

4. Figure 8 shows the variation of the observed daily production rates for all months of the TSS from May 2012 to April 2013. For calculating the average daily production, non-sunshine days and remarkable very low-sunshine days due to cloud, rain etc. are not considered in the calculation. The maximum water output is found in the month of April. The output is higher in the summer season (March-June), then gradually it decreases in the rainy season (July-October) and then again it increases in winter season (December-February). The pattern of water output shows clearly the monthly and seasonal variation of production of distilled water. The maximum daily production is observed in April as 4.26 lit/m^2-day (0.98 lit/day). The average daily production rate for the year is estimated as 2.10 lit/m^2-day (0.48 lit/day).

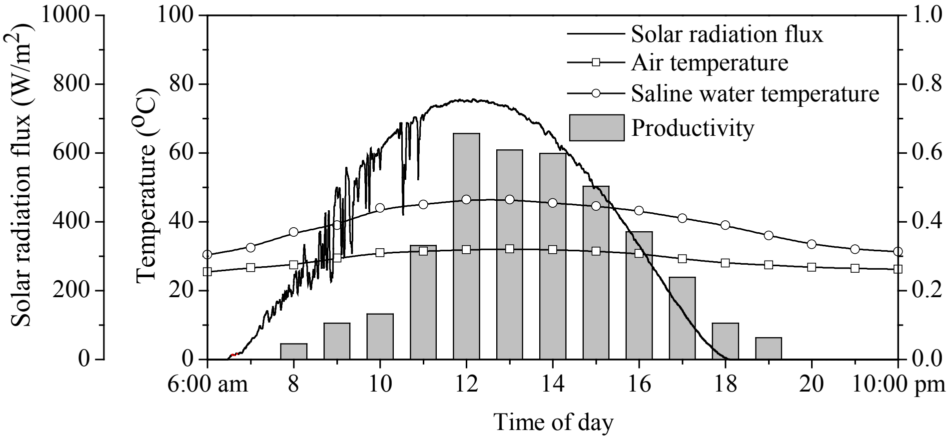


Figure 7: Observed diurnal variations of ambient air temperature, radiation flux and production fluxes for the TSS at KUET, Khulna for April 25 of 2012.

Table 4: Maximum and average daily production rate in lit/m^2-day for each month from May 2012 to April 2013

	May	June	July	August	September	October	November	December	January	February	March	April
Maximum	4.14	3.58	3.33	3.12	3.02	2.53	1.95	1.84	2.24	2.33	3.76	4.26
Average	3.09	2.8	1.80	1.63	1.38	1.33	1.22	1.55	1.97	2.04	3.16	3.22

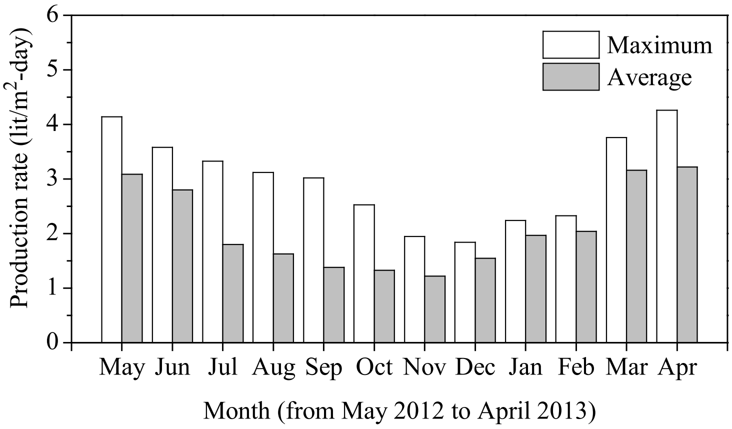


Figure 8: Variations of the observed daily production rates for the TSS from May 2012 to April 2013

Figure 9 shows the relation between the daily production flux, m_{pd} and dialy solar radiation flux, R_{sd} in Khulna for the period May 2012 to April 2013, respectively. From the figure, it is seen that m_{pd} almost varies linearly with R_{sd}. The correlation between the two variants may be expressed by the regression **Equation (1).** Hence, it is clear that radiation is one of the main factors that affect the productivity of the still.

$$m_{pd} = -0.479 + 0.194 R_{sd}\,; \quad (r^2 = 0.88) \tag{1}$$

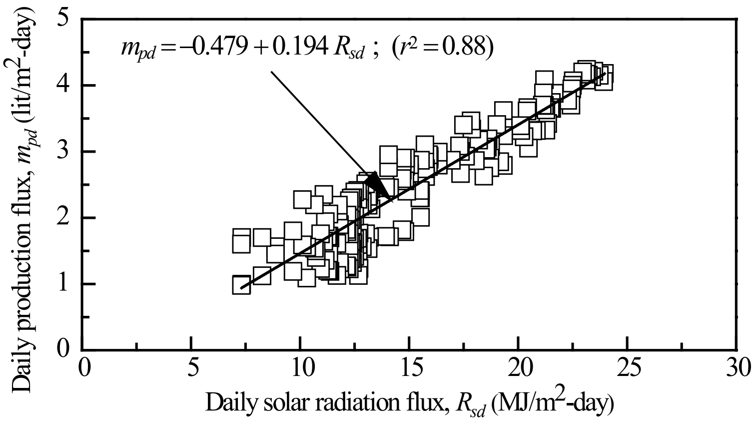


Figure 9: Variations of daily production flux for the TSS with daily radiation flux from May 2012 to April 2013.

Table 5 shows the cost estimation of tubular solar still. The initial cost of each tubular solar still was estimated as Tk. 268.00 and the total cost throughout the design life is estimated as Tk. 468.00. Production cost of water from the TSS is estimated as 0.27 Tk./lit.

Number of TSS needed for a family

Number of TSS required for a family can be calculated as follows:

$$\text{Number of TSS needed} = \frac{\text{No. of family member} \times \text{water requirement (lpcd)}}{\text{Average daily production rate from a TSS (lit/day)}}$$

For a family of 4 members and assume water requirement (lit) for each person per day as 2 lpcd,

Total nos. of TSS required is = $\frac{4\times2}{0.480}$ = 16.7 ≈ 17 nos.

Table 5: Cost estimation of tubular solar still (Design life = 10 years)

Sl. No.	Item Description	Unit	Rate (Tk.)	Quantity	Amount (Tk.)
1	G.I. Wire	Kg	80	0.5	40
2	Wire Mesh	m^2	100	0.5	50
3	Cement	Kg	9	10	90
4	Sand	m^3	1500	0.02	30
5	Polythene	m^2	20	1.5	30
7	Black Oxide	Kg	180	0.1	18
8	Miscellaneous	--	--	--	10
				Initial Cost =	**268**
9	Maintenance cost @ Tk. 20 per year	--	--	--	200
				Total cost throughout design life =	**468**
Design life of a TSS		= 10 year			
The average daily production		= 2.10 lit/m^2-day			= 0.48 lit/day
Total production of water in the design life		= 0.48×10×365			= 1752 lit
Production cost of water		= 468 /1752			= 0.27 Tk./lit

6. CONCLUSION

In this research, perfomance stduy of six PSF's from Paikgacha, a typical arsenic contaminated Upazilla in the coastal belt of the south-western region of Bangladesh and efficacy of low cost TSS have been conducted to justify whether a TSS could be an appropriate solution for such regions to meet the fresh water demand for drinking purpose or not. The results show that the water quality parameters for the selected PSF's vary with the World Health Organization (WHO) and Bangladesh Standards (BDS) allowable limits. The quality of the raw water is found extremely poor. Turbidity, dissolve solids and total solids values of both raw and treated water samples are found within the allowable limits. In most cases, values of suspended solids are exceed the WHO and BDS allowable limit, although there is some improvement of suspended solids observed in treated water. The total coliform, E.coli and BOD_5 values represents that the treated water from the PSF is unsuitable for drinking. It is concluded that suspended solids, total coliform, E.coli, color and BOD_5 are the main problem in the selected PSF's. Therefore, the performance of the existing PSF are found not satisfactory; as a result, the local peoples are frequently suffers by several water born diseases. This study also reveals that performances of PSFs are declining due to lack of technical knowledge, awareness and reluctance among the beneficiaries. It is concluded that the problems in these areas can be solved by using low cost household solar unit system. The construction, operation and maintenances of TSS are simple and environment friendly and also the construction and the water prduction cost of TSS are found very low. It is concluded that the TSS could be an appropriate solution for such regions to meet the fresh water demand for the drinking purpose. It is also a best technique to fulfill the demand of fresh water for drinking purpose for single household in remote and coastal areas. The results presented in this study give clear information to understand the behavior of production rate, other parameters and correlations of daily output for new TSS.

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REFERENCES

Ahmed, M.F. and Rahman, M. (2007). Water Supply & Sanitation, Rural and Low Income Urban Communities, 3rd Edition, ITN-Bangladesh, 301-303.

DPHE-UNICEF. (1989). A Report on the Development of a Pond Sand Filter.

Nickson, R.T., McArthur, J.M., Shrestha, B., Kyaw-Nyint, T.O., and Lowry, D. (2005). Arsenic and other Drinking Water Quality Issues, Muzaffargarh District, Pakistan. Appl Geochem, Vol. 20, 55–68.

Ramkumar, T. and Anita, M. I. (2010). Hydrogeochemical Quality of Groundwater in Vedaraniyam Town, Tamil Nadu, India. Journal of Environmental and Earth Sciences, Vol. 2, 44-48.

Water Corporation. (2000). Desalination: A viable resource, A Strategic review of Desalination possibilities for Western Australia, Australia.

World Health Organization (WHO). (1996). Guidelines for Drinking-water Quality, 4th Edition, 176-189, 222-230.

STUDY OF ROAD DEPOSITED SEDIMENT BUILD-UP PATTERN

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1. ABSTRACT

This paper discusses the outcomes of a research project on road deposited sediment (RDS) build-up on urban road surfaces. RDS build-up was investigated on road sites belonging to residential and industrial land use types. Although roads share a small portion of urban land use, they are producing more pollutants due to its use. Often pollutants attached with RDS were transported with rainfall events to the nearly water bodies, thus it makes receiving bodies polluted. The RDS build-up pattern, if known, can be an indicative to pollutant load derived from the road-traffic environment. In order to control the urban diffuse pollution, it is an important process that need in depth understanding through investigation. As build-up varies with climate, land use, atmospheric deposition primarily, this pattern therefore illustrates highly dynamic and site specific nature. Samples of sediment build up on urban roads (Agrabad and Baizid) in Chittagong city were collected for different antecedent dry days (ADD) by using brush and dustpan from a 1 m^2 area cleaned prior to collection. Samples were collected from each of investigation road sites were analyzed for build up with ADD. The samples collected were also analyzed for particle size distribution to see the variation of sediment sizes exist for different ADD. Based on the results obtained, it has been seen that the rate of build-up was initially in the range of 16 to 17 g/m^2/day and 162 to 170 g/m^2/day, respectively for residential and industrial road surfaces. Later on, the rate of build-up was found in the decreasing order. Considering the build-up pattern with ADD, it was noted that, build-up patterns were different for two sites, illustrating very much site-specific nature rather than general. From the particle size distribution (PSD), it is revealed that the re-distribution of sediment sizes occurred with antecedent dry days (ADD), describing that particle sizes increased with the increment of ADD. Furthermore, mathematical equations of RDS build-up patterns for two sites were formulated to estimate probable RDS load on road surfaces by knowing ADD only.

Keywords:*Road deposited sediment, Antecedent dry days, Build-up, land use, Chittagong.*

INTRODUCTION

Roaddeposited sediment (RDS), inappropriately named "road dust", is recognized as an important source of contamination in urban environments. Road sediments are a complex environmental media. Their composition reflects inputs from a variety of sources, including water transported material from surrounding soils and slopes, dry and wet atmospheric deposition, biological inputs, road surface wear, road paint degradation, vehicle wear(tires body, brake linings, etc.), vehicle fluid and particulate emissions, and inputs from the wear of sidewalks and buildings (Pal et al., 2011).

Road is a vital element of urban land uses. Although roads comprise a small portion of urban land use, they are found to be a significant pollutant sources and sink of airborne pollutants deposited on surface. Among the pollutants present in RDS, heavy metals, organic and inorganic compounds are primarily found to be persistent in nature and very toxic for aquatic lives (Shaheen, 1975). In comparison to developed world, the road drainage system in Bangladesh is completely absent, and it has been seen that runoff generated from the road surfaces is directly discharge to the nearby water bodies. Moreover, in absence of the severity of pollutants level, this area was often neglected and not studied in detail yet in Bangladesh though a lot may be found elsewhere. Before assessing the pollution risk from road surfaces, it is necessary to have in-depth knowledge on the build-up and wash-off processes. A rigorous review of the literature it has been found that RDS build-up pattern is highly variable and site-specific in nature (Ball et al., 1998; Egodawatta et al., 2006; Pal et al., 2011). These studies found that build-up primarily varies with antecedent dry days (weather pattern), traffic volume, road condition, surrounding land uses, road cleaning methods etc. Therefore, the available build-up patterns unlikely produce a true scenario for different region and sites. In particular to Bangladesh there is no such information available to the present knowledge of authors.

Within the above context, this study aims to examine the RDS build-up pattern taking two different sites in Chittagong city. As noted by Sartor et al. (1974) pollutant build-up on catchment surfaces is a dynamic process, such as pollutants accumulated on a road surface over the days may be removed due to the influence of re-distribution by wind and vehicular induced re-suspension and redistribution over antecedent dry days, the rate of build-up isvaried as well. All these factors urge the site specific detail study prior to applying the existing data and information in literature. It is hoped that the equation derived for RDS build up pattern for this study can be useful to predict RDS load for sites with similar characteristics elsewhere.

MATERIALS AND METHODS

Study Site

The study area is road site in Chittagong city. As stated earlier, RDS build-up pattern is highly site specific, two different sites were selected. These wereresidential road in Agrabad and industrial road in Baizid in Chittagong. The street surfaces are made up of asphalt and are in good condition at both sites, details of which are given in next heading. The roads are maintained by Chittagong City Corporation (CCC) and it has been evident that there is no street cleaning (sweeping) program carried out by CCC. Thus, RDS and associated pollutants, which are deposited therein, are carried by rain directly into the nearby water bodies (ponds, khal, lakeetc.). The description of study sites are as follows:

AGRABAD RESIDENTIAL ROAD SITE:

This site in Agrabad is comprised with residential land uses along with a very small portion with commercial land use (presence of some corner shops). It is located in southern part of the Chittagong city, close to the city's seaport.The road is used by light commuters (Rickshow, CNG Autos, Cars, motorcycle, Van primarily) and traffic count at site was found as 400 vehical per day. The site is surrounded by trees, open spaces and residential buildings of typically 6-storied high.

BAIZID INDUSTRIAL ROAD SITE:

The Baizid is an industrial area having a lot of different types of heavy industries such as steel mills, cement factories, garments etc. Baizid site has far 4 lane traffic with average daily traffic is 10000. The presence of industries and people movement, traffic needs to undergo more stop and start at this site. Moreover atmospheric deposition from industry derived ashes was also noticed. This site is also under traffic signal controlled site also with road marking which were absent in Agrabad residential sites.

RDS COLLECTION

A range of sampling technique was discussed in the literature with their relative advantage and disadvantage (see Deletic and Orr, 2005). Researchers often used combinations of the available techniques in order to enhance the collection efficiency. For example, brushing or sweeping of road surfaces is generally efficient in collecting relatively larger particles, while vacuum collection system is advantageous for collection of finer sediment (see Robertson et al., 2003; Deletic and Orr, 2005). Considering the ease of use and nature of sediment, it was decided to use brushing technique. Different brush and dustpan were used for different locations to avoid cross contamination of the sample.

The primary variable which formed the focus of the build-up investigations was antecedent dry period. Depending on the weather pattern antecedent dry days available for study RDS build-up pattern are found to be varied from 5 days to 21 days in literature (see, Sartor et al., 1974; Ball et al., 1998; Egodawatta et al., 2006; Pal et al., 2011). However, it was not clear from past studies the optimum duration for undertaking pollutant build-up investigations due to variations in investigation technique and site conditions. It was decided to continue build-up investigation up to at least 7 antecedent dry days for this research. The antecedent dry periods considered were 1, 2, 3, 5 and 7 days. Sample collection was undertaken on 1.0m x 1.0m road surfaces plots. These plots were selected from the curb end of one side of the road keeping approximately 1m distance. Plots were initially cleaned by repeated brushing. At the end of each antecedent dry period, RDS were collected from each plot of two different sites maintaining similar pressure of sweeping. Brushing was done several times in perpendicular directions in order to ensure that all the particulate material was collected. A permanent painting was used to locate the plot boundary during sample collection (see Fig. 1). The pressure maintained for sweeping was in such that was not dislodged road material. The detail procedure can be found in Pal et al., (2011). After collection, sample was transferred to a clean plastic bag with date and tag from each site from dust pan prior further analysis. During the period of sample collection, no street sweeping was seen to be carried out from the sampling sites.

Figure 1: Collection of RDS from Agrabad Residential Area in Chittagong

CHEKING SAMPLING EFFICIANCY

The sampling efficiency of the brushing system was tested under a standard condition using a 1m x 1m clean road surface prior to sampling of RDS. Similar types of pollutant collected from road. A solids sample of 100g mass and known particle size distribution was spread evenly on the surface using a straight edge and a fine brush. Care was taken to ensure that none of the solids spilled over the edge. The solids sample spread on the surface was collected using the brush. The procedure adopted was to brush the surface three times in perpendicular directions. The collected sample was weighed. The total weight recovered was 94.57g for the brush used in residential road and 92.13g for the brush used in industrial road.

LABORATORY ANALYSIS

As stated earlier, after collecting the build-up samples, they were preserved in plastic bags. Then collected samples were transported to the 'Environmental Engineering Laboratory' at CUET for further analysis. Sample handling and preservation was undertaken as per laboratory standard. First of all, sample was weighted using weighing machine with a accuracy of 0.01g. Later on sieve analysis was performed to indentify the gradation of RDS for different dry days from each site. The samples from each sieve sizes were then preserved with additional tag in the laboratory for analysis of heavy metals that are not presented and discussed in this paper.

RESULTS AND DISCUSSION

RDS Build-up Load and Pattern

Table 1 presents the data of RDS collected from each site (Agrabad R/A and Baizid I/A) for 1, 2, 3, 5 and 7 antecedent dry days (ADD). The build-up load range from 16.2 gm/m^2 to 84.3 gm/m^2 and 166.3 gm/m^2 to 1029.1 gm/m^2 for Agrabad R/A site and Baizid I/A site, respectively, for 7 ADD. Comparing both sites, it is evident that the build-up load from Baizid I/A road site is an order magnitude higher that of Agrabad R/A road site. The variation is not unexpected rather consistent with other studies elsewherefor two completely different sites with different land use pattern along with site surrounding variability, traffic volume and road surface condition and control system present on road (Ball et al., 1998; Egodawatta et al., 2006; Pal et al., 2012). From the close observation of road sites, it was further identified that wind effect is much less due to tall factory buildings located at Baizid I/A site compared to Agrabad R/A site with low height residential building with open space.

Table1: RDS load found from study sites for different antecedent dry days

Land use	RDS (gm/m^2)				
	1day	2 days	3 days	5 days	7 days
Agrabad Residential Area	16.2	34.8	51.8	76.6	84.3
Biazid Iindustrial Area	166.3	328.4	541	857.1	1029.1

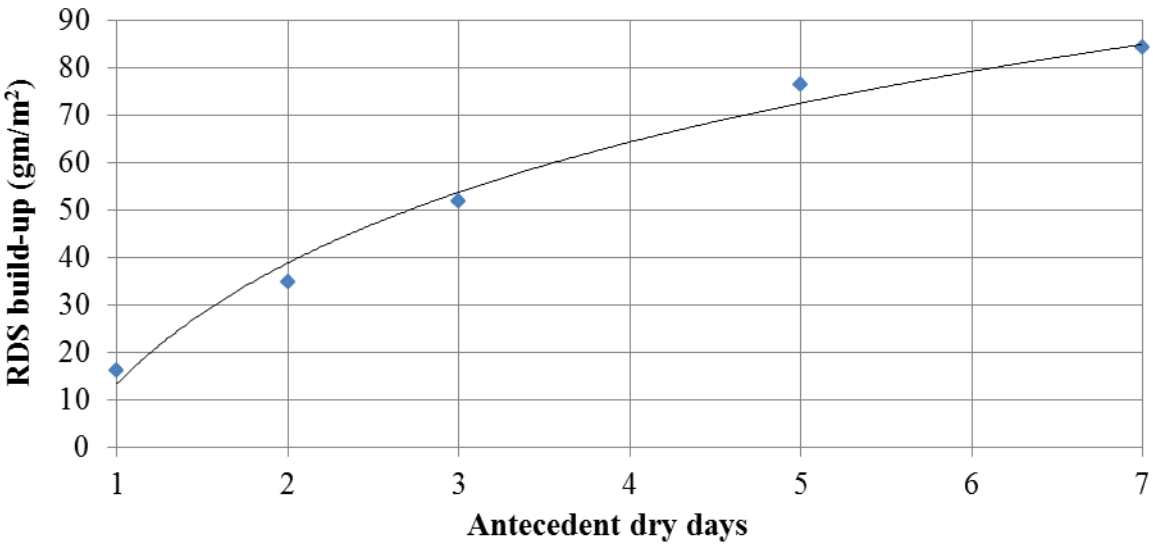

Figure 2: RDS build up pattern found for Agrabad R/A site

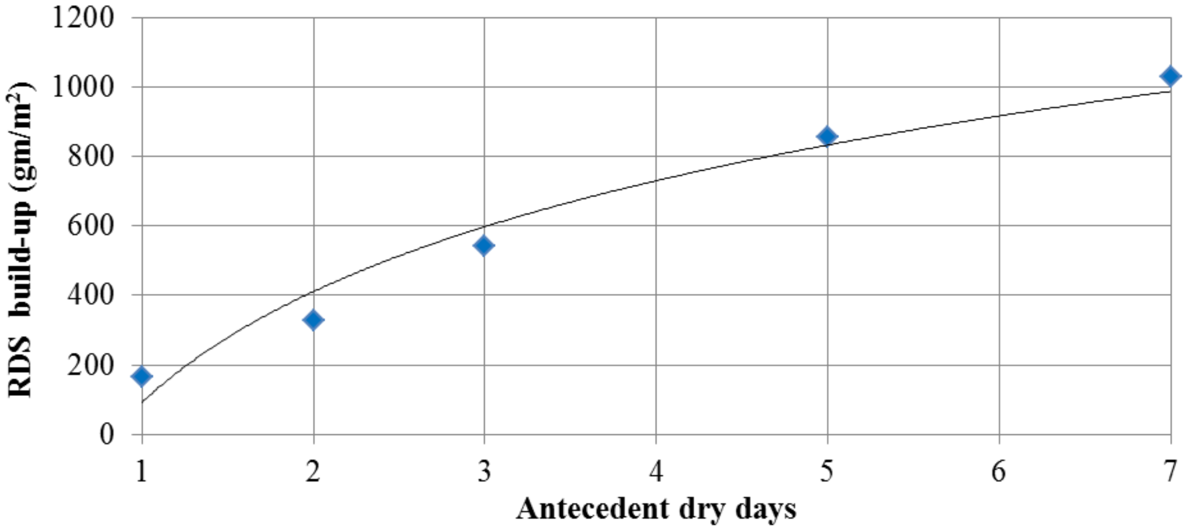

Figure 3: RDS build up pattern found for Biazid I/A site Area

RDS build-up pattern was plotted to discover the rate of build-up pattern which are shown in Fig. 2 and Fig. 3. Analysis of build up data revealed rapid build-up during the initial period after site cleaning while rate of increment is gradual after 5 ADD. The rate of build-up was initially in the range of 16 to 17 g/m^2/day and 162 to 170 gm/m^2/day for residential area site and industrial area site, respectively, and was later decreased to when the antecedent dry days increased as to 5 to 12 gm/m^2/day and 80 to 120 gm/m^2/day, respectively for the above sites. The range of variation is bit high compared to the values found for other sites elsewhere (for example, Deletic and Orr, 2005; Robertson et al., 2003; Ball et al., 1998). Different weather pattern and road maintenance along with vehicle types and ages may likely the factor for differences in results. Considering the rapid variation in build up during the initial period was to be due to the higher impact of anthropogenic activities on road surfaces such as traffic, while the reduction in the rate of build up with the higher ADD may be considered to be due to the influence of pollutant redistribution. The concept is justified the gradual build up pattern observed on road surfaces elsewhere.

Mathematical Formulation of RDS Build-up Pattern

In order to replicate and quantify RDS build-up load from sites with similar characteristics, the observed build-up data as seen in Fig. 2 and Fig. 3 were analysed for several mathematical functions available and suggested by Ball et al. (1998). It has been found that the pattern was described moderately well by using power law function for both the site from the available functions studied by Ball et al. (1998). However, for this study build-up pattern can be better explained by logarithmic function. Authors believe more data for longer ADD (only 7 days were considered for the present study) is required to reach a concrete conclusion. However, build-up coefficients obtained by using power law function for this study were compared with the values obtained by Egodawatta et al. (2010). The results obtained are presented in Table 2. The power law function used was in the form of:

$$B=aD^b$$

where B is the RDS build-up load in gm/m^2, D is the number of antecedent dry days, a and b are coefficients.

Table 2: RDS build-up pattern coefficients obtained for study sites

Study Site	a	b	Reference
Residential area	18	0.866	Present study
Industrial area	172	0.964	
Townhouse Region, Gumbeel Court, Australia	2.90	0.16	Egodawatta et al. (2006)
Single detached housing regions, Lauder Court and Piccadilly Place, Australia	1.65	0.16	

The coefficients are found greater in magnitude than that of Egodawatta et al. (2006), as seen in Table 2. The difference is likely due to different attributes, such as road surface condition, differences in weather patterns, surrounding land uses, traffic volume etc., and signify that this process is highly variable with local/site specific influences.

Particle size distribution of RDS

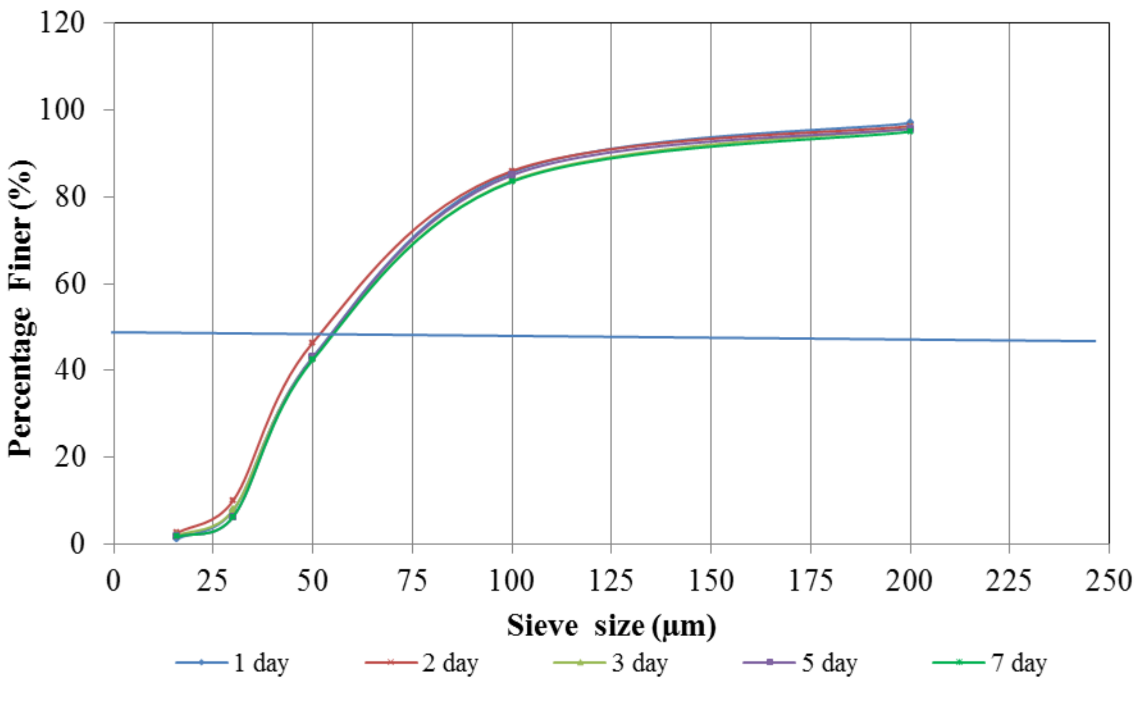

Figure 3: Particle size distribution of RDS collected from Agrabad R/A site

Furthermore the particle size distribution of collected RDS from both the sites and for 7 days were carried out to see the variation of sizes occurred during dry days. The mean RDS size (d_{50}) was found to vary in the range of 52 µm to 56 µm showing vary little difference for Agrabad residential site, while it ranged 62 µm to 75 µm for Baizid industrial site, as seen in Fig. 3 and Fig. 4. It is anticipated that the particle sizes increased as ADD increased. The d_{50} obtained for this study is consistent with the values obtained elsewhere (Pal et al., 2012; Sartor et al., 1974). The changes in particle sizes are likely to the redistribution and re-composition after deposition. Due to the re-suspension of finer sediment due to wind and vehicle induced turbulence, usually at the curb site of the road relatively bigger sizes are found as days progress (Pal et al., 2012).

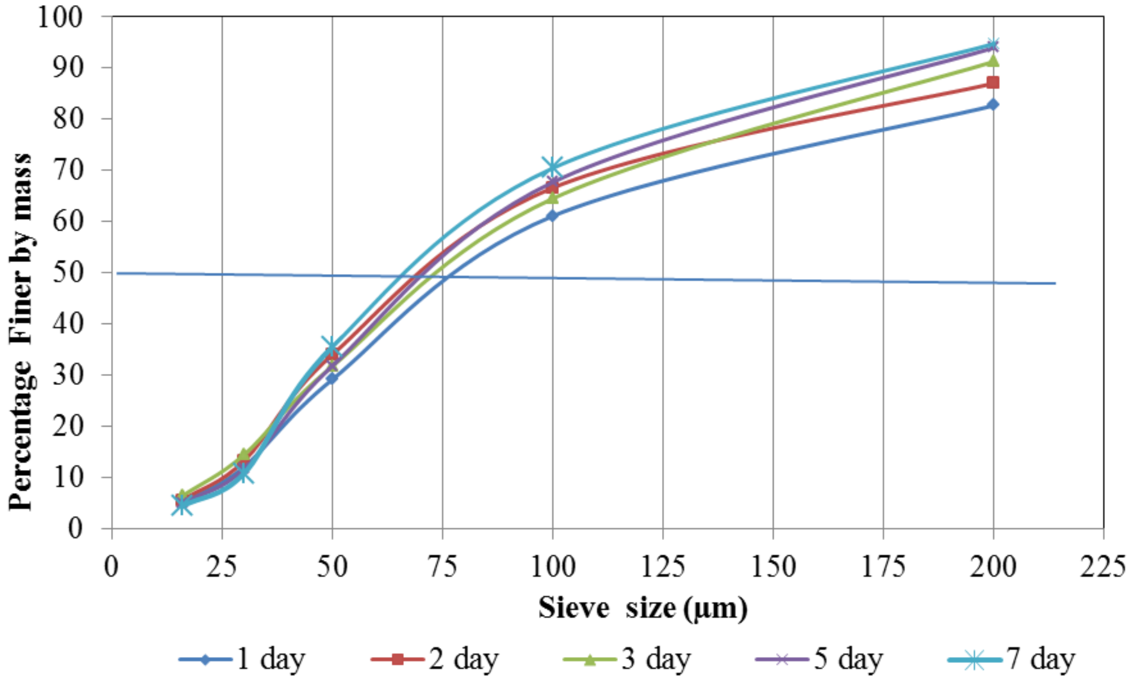

Figure 4: Particle size distribution of RDS collected from Baizid I/A site

CONCLUSION

Analysis of RDS build-up on road surfaces from residential and industrial site in Chittagong city that is for the first study on this type leads to the following conclusions:

- Two road sites investigated showed significant variation in terms of build-up load. This suggested a highly variable nature of build-up influenced by land use, traffic volume and road surface primarily.
- The rate of build-up was initially high and then gradually decreased to reach almost a constant level and the variation is consistent with literature. The build-up coefficients obtained for this study were estimated that could be used for other sites with similar characteristics elsewhere.
- Particle size distribution of RDS collected showed sizes are bigger for larger antecedent dry days because of redistribution during their residence time on road surfaces.

REFERENCES

Ball, J.E., Jenks, R. and Aubourg, D. (1998). An assessment of the availability of pollutant concentrations on road surfaces. Science of the Total Environment 209, 243-254.

Deletic, A. and Orr, D.W. (2005) Pollution buildup on road surfaces. Journal of Environmental Engineering131, 49-59.

Egodawatta, P. and Goonetilleke, A. (2006). Characteristics of pollution build-up on residential road surfaces. Paper presented at the 7th International conference on hydro-science and engineering held during 10-13 September 2006, USA

Pal, S.K., Wallis, S. and Arthur, S. (2012). Emission patterns of traffic-related metals and associated contamination in road deposited sediment. Int. J. Global Environmental Issues 12, 229-241.

Pal, S.K., Wallis, S. and Arthur, S. (2011). Assessment of heavy metals emission from traffic on road surfaces. Central European Journal of Chemistry 9, 314-319.

Robertson, D.J., Taylor, K.G. and Hoon, S.R. (2003) Geochemical and mineral magnetic characterisation of urban sediment particulates, Manchester, UK. Applied Geochemistry 18, 269–282.

Sartor, J.D., Boyd, G.B. and Agardy, F.J. (1974) Water pollution aspects of street surface contaminants. Journal of the Water Pollution Control Federation 46(3), 458-465.

Shaheen, D.G. (1975) Contributions of urban roadways usage to water pollution. *Report No. EPA-600/2-75/004,* Municipal Pollution Control Division, Office of Research and Development, US Environmental Protection Agency: Washington DC, USA

LABORATORY INVESTIGATION OF SOFT SOIL IMPROVEMENT BY CEMENT COLUMN

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ABSTRACT

Due to scarcity of land for rising population in a country like Bangladesh, it is necessary to improve the soft soil to face the challenges of this problem. Soft soils are generally labeled as 'problematic' because of poor resistance to deformation and very low bearing capacity. Thus, improvement of the weak properties of soft soil is required, which can be achieved by adopting cement column as one of the soil improvement techniques. This paper aims to define the effect of cement column in improving soft soil and their installation technique by laboratory investigation through small scale test. To check the degree of improvement of soft ground due to the installation of cement column is the main objective of the study. A mixing machine, fabricated locally as a part of this study, is used here to provide cement column in the soft soil. ASTM D2166 is used to determine the unconfined compressive strength of the reconstituted soil, which is used further to determine the bearing capacity of the soil media. The "Universal Testing Machine" is used for determination of load-settlement behavior of the cement column improved ground. Finally, from the experimental investigation it was observed that the bearing capacity of soft ground can be increased significantly through the installation of cement column.

Keywords: *Soft soil, Bearing capacity, Cement column, Reconstituted soil, Unconfined compressive strength.*

1. INTRODUCTION

The rapid growth in the infrastructure of urban and metropolitan areas in most countries of the world has resulted in non-availability of suitable locations. Accordingly, the marginal ground and reclaimed land with poor soil conditions; especially in coastal regions and low land areas are becoming more attractive for development. Weak deposits are very common along the coastal region. Most of the marine deposits are of recent origin and have not undergone much consolidation. As a result, they have low shear strength or high compressibility. Even some of the land deposits, particularly alluvial deposits along the river belt have loose silt/sand to a large depth. Even man made deposits such as mine back-fill or land reclaimed by filling can have inadequate strength properties requiring ground improvement.

Deep mixing columns using cement mixed in-situ with soft soil to stabilize soft clay and organic soil are commonly used in Sweden, Finland, Norway and Japan. These methods of soil stabilization have gradually been improved since 1967 when Mr. Kjeld Paus patented the method and subsequently new techniques were invented. In Malaysia, soil improvement is very important as the country is abundant with weak soil that is unsuitable for construction works. Weak soil such as Alluvium Clayey is abundant throughout Malaysia, constituting 70% of 5000km of the country's coastline ranging between 20 to 40 meters in soil thickness. Thus, ground settlement is an issue due to low bearing capacity of soft clay material which can cause problems such as low stability and excessive settlement (Nur et al., 2011). Not only those countries but also in Bangladesh it is a problem because it is a land of delta formation with alluvial deposition. The method has gradually been improved in Scandinavian countries since the 1970s (Broms 1999a). Since the end of the 1990s, it has been the most commonly used method in Sweden for stabilizing soft soils. According to Swedish practice, stability calculations are based on the assumption that the columns and surrounding soft soil behave as a composite material (Carlsten & Ekström, 1995). Excavation support using deep soil mixing technology evolved from the early 1970's Japanese practice, in which single soil-cement columns were created to support excavations and act as cutoff walls. The behavior of cement column by deep mixing method has been investigated experimentally by Miyake et al. 1991, Hashizume et al. 1998 and Kitazume et al. 1999. The increase in strength with time of surrounding clay adjacent to soil-cement columns was experimentally and numerically studied by Miura et al. 2001 and by Shen and Miura 1999. The factors controlling in-situ strength of soil-cement columns have been investigated by a full-scale test (Horpibulsuk et al., 2000). The recent laboratory investigation on the strength development in cement admixed clay at various conditions of cement content and water content is presented by Miura et al. 2001 and Horpibulsuk and Miura 2001. Horpibulsuk et al. 2001 have proposed interrelationship among water content, cement content, curing time and strength of cement admixed clays. The application of deep mixing technique to reduce settlement of an embankment in Thailand was successfully done by Bergado et al. 1999.

In this study, the degree of improvement of soft ground due to the installation of cement column is investigated in the laboratory. It was observed that the bearing capacity of soft ground can be increased significantly through the installation of cement column.

2. SOIL CEMENT COLUMN TECHNIQUE

This method is commonly known as deep mixing method or admixture method. Lime or cement columns, where quicklime or dry cement are mixed in situ with soft soil as shown in Figure 1, are common in Sweden and Finland, to stabilize soft clay and silt as well as organic soils. The method has gradually been improved and new applications have been found. Lime or cement columns have mainly been used to increase the stability and to reduce the settlements of road and railroad embankments and to increase the stability of trenches for sewer lines, water mains and heating ducts. New efficient machines have been developed for the installation of the columns. The diameter and the length have gradually increased and the time required for the installation of the columns has been reduced significantly as well as the costs.

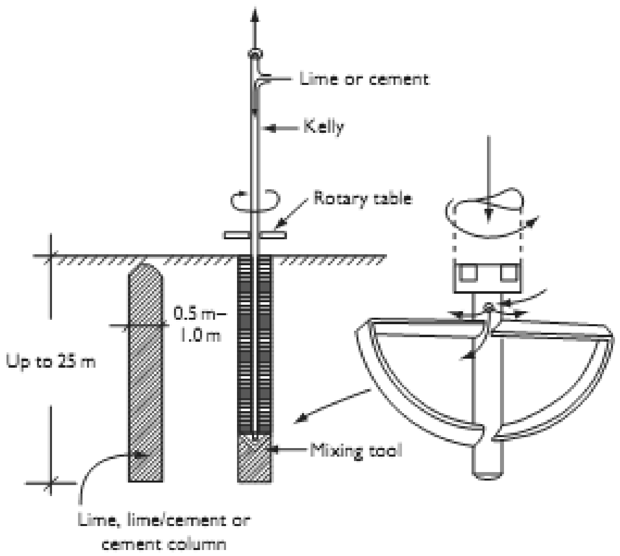

Figure 1: Installation of lime or cement column.

Numerous projects have incorporated deep mixing for excavation support and reduce settlement. One of the first major applications of cement column as deep soil mixing for excavation support in the United States was the Wet Weather Storage Basin for the East Bay Municipal Utility District (EDMUD) project in Oakland, California constructed in 1990 (Taki and Yang, 1991). One of the largest projects in the United States involving deep soil mixing technology is the Boston Central Artery and Tunnel (CA/T) project (O'Rourke and O'Donnell, 1997a and 1997b; O'Rourke et al., 1998; O'Rourke and McGinn, 2004). Yang, 2003; states that the improved engineering properties of the stabilized soils are governed by a number of factors including soil type, slurry properties, mixing procedures and curing conditions.

The main areas of soil mixing applications are as follows, with the countries in parentheses indicating their most extensive use so far:

- Foundation support (Japan, Scandinavia, US, France, Poland)
- Retention systems (Japan, US, China, Southeast Asia, Germany)
- Ground treatment (Japan, US, Finland, Sweden, Southeast Asia)

- Liquefaction mitigation (Japan, US)
- Hydraulic cut-off walls (Japan, US, Germany, Poland)
- Environmental remediation (US, UK).

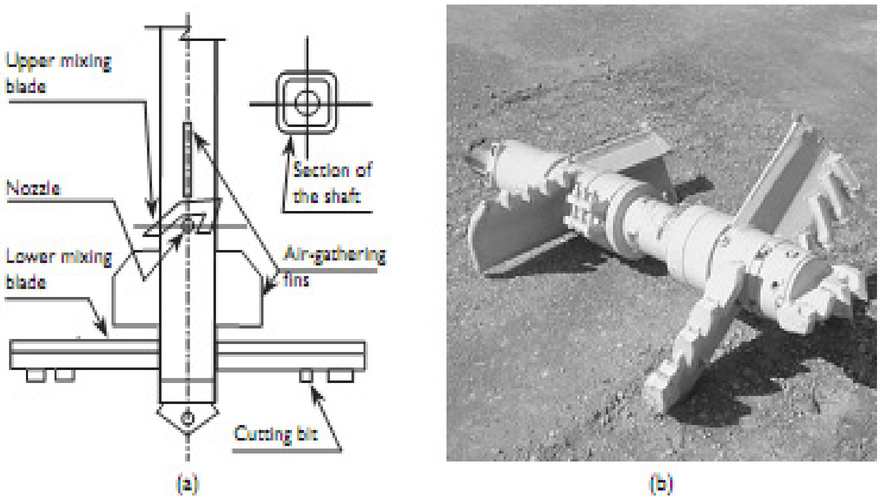

Figure 2: Mixing tools of the DJM method. (a) Construction scheme (DJM Association, 2002); (b) Recently used single mixing tool of 1.0m diameter

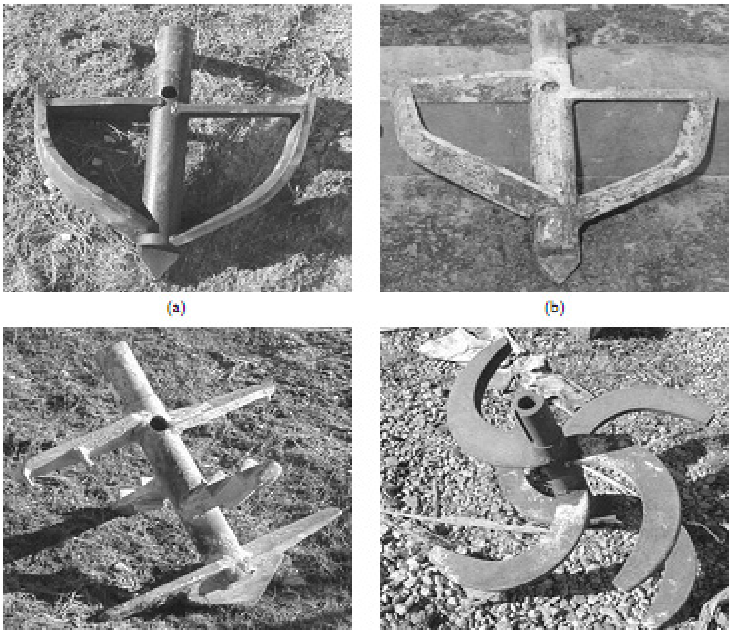

Figure 3: Selected mixing tools of the Nordic method: (a) SD 600 mm; (b) modified SD 600 mm; (c) PB3 600 mm; (d) peat bore 800mm (courtesy of LCM)

In the deep soil mixing process, admixtures/binders are introduced into the in-situ soils throughout the treatment depth and mixed thoroughly using large diameter single or multiple-shaft mixing tools to form columns or panels of improved material (Figure 4). The mix-in-place columns can be up to 1m or more in diameter. Typical admixtures are cement and lime, but slag / flyash and/or other additives can also be used.

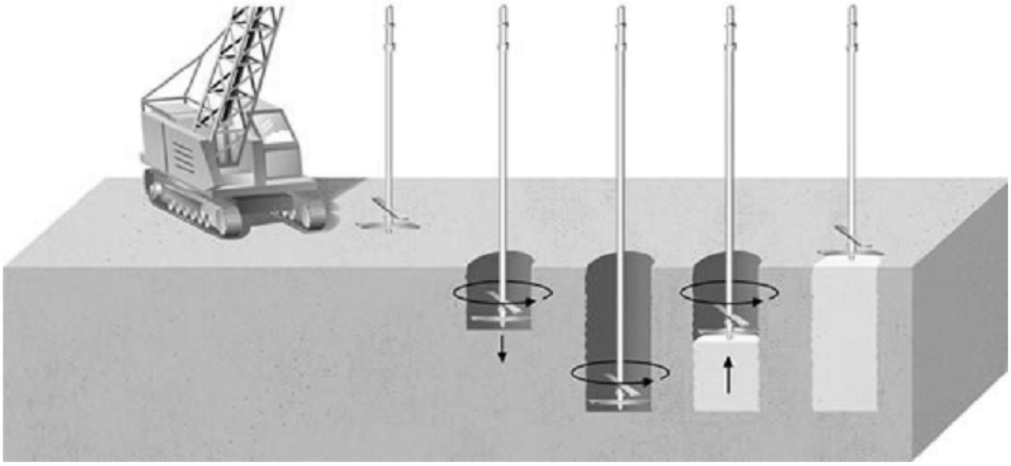

Figure 4: Schematic showing overall process of Deep Soil Mixing.

For both wet as well as dry Deep Soil Mixing, quality control during execution is important to ensure uniform improvement of the soil and to ascertain the required amount of binder has been mixed uniformly over the entire depth of treatment. For this purpose, the mixing units are equipped with automated computerized recording devices to measure the real-time operating parameters such as depth of mixing tool, volume or weight of binder used, flow rate of grout, rotation speed and rate of penetration and withdrawal. After allowing for sufficient curing period (typically, 3 to 4 weeks), the mixed columns can also be tested using single/group column plate load tests, unconfined compressive strength tests on cored/backflow samples, visual examination of exposed columns, etc.

3. MATERIALS AND METHODOLOGY

3.1 Properties of Clay (With and Without Cement)

Table 2: Properties of foundation soils

Property	Value	
	Without Cement	With Cement
Liquid Limit	31%	36%
Plastic Limit	21%	26%
Moisture Content	35%	25%

3.2 Formation of Clay Bed

To form clay bed the following steps were taken (Figure 5)-

- Clay sample was collected from KUET.
- A 18" diameter and 20" height circular drum was made.
- The bottom part of the drum was made hole for drain out water.
- A geo-jute was in the bottom part of the drum that only permit to flow water not clay.
- Then full fill the drum with clay sample to get a circular clay bed.
- Make a circular slab to distribute the surcharge load uniformly to the clay bed.
- Now kept it 28 days for drainage with surcharge.
- Finally clay bed was prepared.

Figure 5: Formation of clay bed

3.3 Installation of Cement Column

For installation of cement column in clay soil the following steps were done-

- A mixing device was prepared which is shown in figure 6.
- After that the mixing device injected into the clay bed by hand rotating.
- In this time the cement was poured in the clay by the device and mixed with the clay.
- The diameter of the column is 6" in diameter which is one third of the clay bed.
- Only 7% cement of the total column is used and mix with the soil.
- Finally the mixing device is lift from the bed by reverse rotating.

Figure 6: Mixing Device for Installation of Cement Column

Figure 7: Load Settlement Measurement

The clay bed was tested in "Universal Testing Machine" to find out the settlement against load for both clay bed i.e; without cement column and with cement column (Figure 7). Then series of data were plotted in a graph to find out the load settlement relationship and finally compared this graph for improvement.

4. RESULTS AND DISCUSSIONS

4.1 Stress-Strain Behaviour of Foundation Soil

From the graph (Figure 8) it was found that the value of deviator stress for sample without cement column is 18 kPa. Whereas, with cement column the value increased upto 80 kPa. From these values the undrained shear strength was found to be 8.75kPa before cement column and after the value was 40.00kPa.

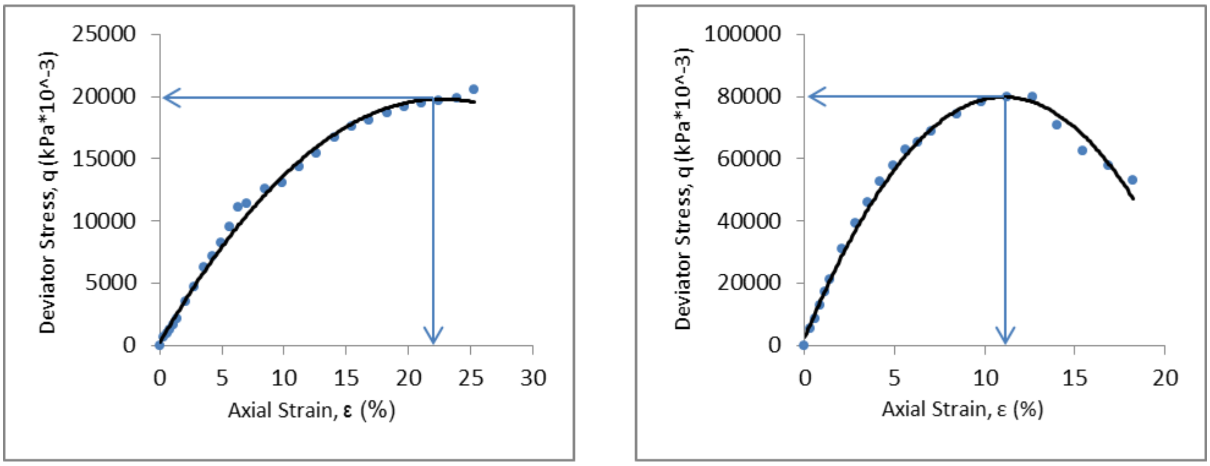

Figure 8: Unconfined Compressive Test with and without cement

4.2 Load Settlement of Foundation

From the load settlement curve shown in figure 9 it was found that the settlement curve for sample with cement column is above than the other one without cement column. It indicates that the bearing capacity of soft soil is increased significantly by using cement column.

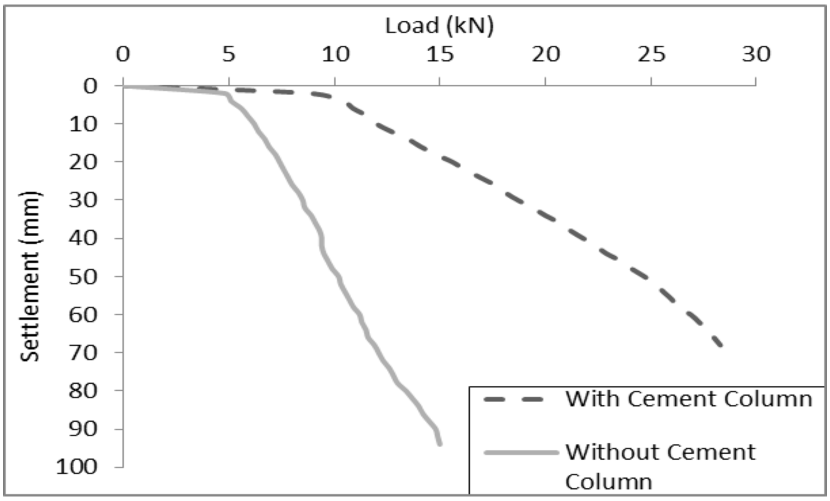

Figure 9: Load Settlement Curve

From graph it was observed that the value of load intensity for cement column is 822.28kPa whereas in case of without cement column it is only 172.38kPa. And the percentage of cement in column is very low, only 7% of volume of soil in cement column.

5. CONCLUSIONS

The modern development of foundation practices, namely ground improvements techniques, to overcome the limitations of the conventional foundation system has been proved to be both technically and economically feasible for the improvement of the marginal sites. Amongst the various ground improvement techniques for improving soft ground conditions, cement column is considered as one of the most versatile and cost effective method. This ground improvement technique has been used in many difficult foundation sites throughout the world to increase the bearing capacity, reduce settlement, and increase the stability.

Based on the laboratory investigation the followings can be concluded-

- The cement column can be applied upto a suitable depth.
- The bearing capacity of soil is improved after installing cement column.

- Also study gives that improved soil by cement column will fail with a greater settlement.

REFERENCES

Bergado, D.T., Ruenkrairergsa, T., Taesiri, Y., and Balasubramaniam, A.S. (1999). "Deep Soil Mixing Used to Reduce Embankment Settlement," Ground Improvement, Vol 3, pp 145-162.

Broms, B.B. (1999a). "Keynote Lecture: Design of Lime, Lime/Cement and Cement Columns". Proceedings of the International Conference on Dry Mix Methods for Deep Soil Stabilization, Stockholm, 13-15 October, pp. 125-153.

Carlsten, P. and Ekström. J. (1995). "Lime and lime cement columns. Guide for design, construction and control". Swedish Geotechnical Society, SGF Report 4:95. pp. 111.

Hashizume, H., Okochi, Y., Dong, J., Horii, N., Toyosawa, Y., and Tamate, S. (1998). "Study on the Behavior of Soft Ground Improved Using Deep Mixing Method," Proceedings of International Conference on Centrifuge 98, pp 851-856.

Horpibulsuk, S. (2001). Analysis and Assessment of Engineering Behavior of Cement Stabilized Clays, Ph.D. Dissertation, Saga University, Saga, Japan.

Horpibulsuk, S. and Miura, N. (2001). "A New Approach for Study Behavior of Cement Stabilized Clays," Proceedings of 15th International Conference on Soil Mechanics and Geotechnical Engineering, Istanbul, Turkey, Vol 3, pp 1759-1762.

Horpibulsuk, S., Miura, N., and Nagaraj, T.S. (2001). "Analysis and Assessment of Strength Development in Cement Admixed Clays," Proceedings of International Conference on Civil Engineering, Indian Institute of Science, India, Vol 2, pp 156-163.

Horpibulsuk s., Miuran N.,Nagaraj T.S. and Koga H. (2002)." Improvement of Soft Marine Clays by Deep Mixing Technique," The Twelfth International Offshore and Polar Engineering Conference, pp 584-591

Horpibulsuk, S., Miura, N., and Nishida, K. (2000). "Factors Influencing

Field Strength of Soil-Cement Column," Proceedings of Geotechnical Engineering Conference, Asian Institute of Technology, Bangkok, Thailand, pp 623-634.

Kamon M. (1991). "Soil Improvement of Soft Clay Ground," GEO-COAST. pp 1043-1053.

Kitazume, M., Yamamoto, M., and Udaka, Y. (1999). "Vertical Bearing Capacity of Column Type DMM Ground with Low Improvement Ratio," Proceedings of International Conference on Dry Mix Methods for Deep Soil Stabilization, pp 245-250.

Miura, N., Horpibulsuk, S., and Nagaraj, T.S. (2001). "Engineering Behavior of Cement Stabilized Clay at High Water Content," Soils and Foundations, Vol 41, No 5, pp 33-46.

Miyake, M., Akamura, H., and Wada, M. (1991). "Deformation Characteristics of Ground Improved by a Group of Treated Soil," Proceedings of International Conference on Centrifuge 1991, pp 295-302.

Nur A., Hafez M.A. and Norbaya S. (2011). "Study of Bearing Capacity of Lime-Cement Columns with Pulverized Fuel Ash for Soil Stabilization Using Laboratory Model," EJGE Vol 16, pp 1595-1605.

O'Rourke, T.D. and McGinn, A.J. (2004). Case History of Deep Mixing Soil Stabilization for Boston Central Artery. Geotechnical Engineering for Transportation Projects, 1, 77-128.

O'Rourke, T.D. and O'Donnell, C.J. (1997a). Deep Rotational Stability of Tiedback Excavations in Clay. Journal of Geotechnical Engineering, pp 506–515.

O'Rourke, T. D and O'Donnell, C.J. (1997b). Field Behavior of Excavation Stabilized by Deep Soil Mixing. Journal of Geotechnical and Geoenvironmental Engineering, pp 516–524.

O'Rourke, T.D., McGinn, A.J., and Dewsnap (1998). Case History of Excavation Stabilized by Deep Mixing Methods, Design and Construction of Earth Retaining Systems, ASCE, Reston, VA,pp 41-61.

Shen, S.L., and Miura, N. (1999). "Soil Fracturing of the Surrounding Clay During Deep Mixing Column Installation," Soils and Foundations, Vol 39, No 5, pp 13-22.

Taki, O. and Yang, D.S. (1991). Soil–Cement Mixed Wall Technique. ASCE Special Conference, Denver, CO, 298–309.

Yang, D.S. (2003). Soil–Cement Walls for Excavation Support. Earth Retention Systems 2003: A Joint Conference presented by ASCE Metropolitan Section of Geotechnical Group, The Deep Foundations Institute, and The International Association of Foundation Drilling, New York City, NY.

CONTENTS (Structural Engineering)

STUDY ON COMPRESSIVE STRENGTH OF CONCRETE USING STONE DUST AS PARTIAL REPLACEMENT OF FINE AGGREGATE.

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ABSTRACT

Concrete is the most undisputable, versatile, and indispensable material which is being used in infrastructure development throughout the world. Due to rapid growth in construction activity, the consumption of concrete is increasing day by day. This results in excessive extraction of natural aggregates. The use of these aggregates is being constrained by urbanization, zoning regulations, increased cost, and environmental concern. Sand is a well-known natural aggregate which is used as a fine aggregate in the concrete. Excessive sand mining could cause the change of local biodiversity, land subsidence and land sliding. Thus, it is becoming inevitable to use alternative materials such as fly ash, crushed rock powder, stone dust etc. as fine aggregates in concrete. Stone dust appears as a problem in stone crushing industry due to lack of effective disposal facilities, but this dust could probably be replaced partially with sand in concrete. Consequently, in this study, the main concern was to optimize the compressive strength of concrete when stone dust was used as a partial replacement of Domar sand and Padma sand. In the experiment, 78 cylindrical specimens of 4 inch diameter and 8 inch height (ASTM C 31) were tested to identify the compressive strength of concrete using universal testing machine. Curing of 7 days was applied for 39 specimens and 28 days curing was conducted for the remaining 39 specimens. As a fine aggregate Domar sand and Padma sand were used separately and afterward these two fine aggregates were replaced partially with stone dust by 10%, 20%, 30%, 40%, and 50%. In the results, it was revealed that concrete made of 30% stone dust with 70% Domar sand get impressive improvement of compressive strength at both 7 days and 28 days curing period. The compressive strength in concrete made of Domar sand and stone dust was improvement up to 28.27 % at 7 days curing and 17.78 % at 28 days curing in compare to the compressive strength of concrete made of Domar sand. Similarly, the compressive strength of concrete was significantly increased for the curing period of both 7 days and 28 days when concrete was made of 40-50 % stone dust and 50-60 % Padma sand. The compressive strength of concrete with stone dust and Padma sand was increased up to 28.72 % and 26.85 % for the curing of 7 days and 28 days respectively, in compare to the compressive strength of concrete made of Padma sand. Therefore, it could be concluded that stone dust could partially be replaced with sand in order to improve the compressive strength of concrete.

Keywords: Concrete; Stone dust; Fine aggregate; Compressive Strength; Curing.

1. INTRODUCTION

Concrete is a widely used construction material consisting of cementing material, fine aggregate, coarse aggregate and required quantity of water, where in the fine aggregate is usually natural sand. The economy, efficiency, durability, moldability and rigidity of reinforced concrete make it an attractive material for a wide range of structuralapplications (Ferguson et al., 1988).Fine aggregate is one of the important constituents that effects the strength of concrete (Sharmin et al., 2006). The gaps of coarse aggregate are filled by the fine aggregate and the gapes of fine aggregate is filled by the binding materials (Aziz,1995). The use of sand in construction results in excessive sand mining which is objectionable. Due to rapid growth in construction activity, the available sources of natural sand are getting exhausted. (Palaniraj, 2003) Also, good quality sand may have to be transported from long distance, which adds to the cost of construction. On the other hand, the modern technological society is generating substantially high amounts of solid wastes both in municipal and industrial sectors; posing an engineering challenging task for this effective and efficient disposal. Hence, partial or full replacement of fine aggregates by the other compatible materials like sintered fly ash, stone dust, crushed rock dust, quarry dust, glass powder, recycled concrete dust, and others are being researched from past two decades, in view of conserving the ecological balance(Keerthinarayana, 2010).

Even though, use of several types of industrial solid wastes like metallurgical waste, glass pieces, fly ash, quarry dust, tyre and rubber waste, crushed concrete waste, sludge and others in making good field concrete is being effectively done at European countries, U.S.A., U.K., and Australia; Asian countries could not gear up to that level to match with those countries. Therefore, resource exploitation and waste disposal problems are currently rocking the sustainable development in those countries(including Bangladesh).

The main constituents of concrete such as sand, stone and water are mainly natural resources. They are not produced in laboratory or in anyindustry; they are obtained from the nature andprocessed to make it perfect for aggregate. For example sand is carried by river water and then collected, andstones are obtained by crushing of bolder using stonecrusher. These resources of engineering materials (sand,stone) are limited and day by day the dependency onthem must be minimized. So some other materials shouldbe introduced by replacing sand and stone. Stone dust isone of such alternative of sand that can fulfill the demandof fine aggregate.

In some cases, natural sand may not be of good quality. Therefore, it is necessary to replace natural sand in concrete by an alternate material either partially or completely without compromising the quality of concrete. Stone dustis one such material which can be used to replace sand as fine Aggregate.

Conservation of natural resources and preservation of environment is the essence of any development. The problem arising from continuous technological and industrial development is the disposal of waste material. If some of the waste materials are found suitable in concrete making, not only cost of construction can be cut down, but also safe disposal of waste materials can be achieved. So in the present paper, an attempt has been made to assess the suitability of stone dust in concrete making. In the laboratory stone dust has been treated as fine aggregate in place of sand used as partial/full substitute to conventional fine aggregate in concrete making.

Every year many buildings are constructed all over the world. Not only the buildings but also many kinds of high value concrete structures like bridge, dam, high rise buildings and industries are constructed. We know that in concrete mix sand is used as a fine aggregate. So every year a huge quantity of sand is used to construct various infrastructures. The good news is that our country is a riverine country so a huge quantity of sand is available in river bed. But if this continues, within the next two or three century, it may causes sand crisis. This may affect badly to the development of our country. Again it also affects the environment badly. The rapid growth of the urban areas may also be affected. To avoid this unexpected situation now we are finding alternative of sand which may cut down the cost of the construction and will make the construction economical. Stone dust can be effectively used as apartial alternative of sand in concrete mix. It is a natural product produced after the crushing of stone particles. This stone dust is like dust with fine stones. Normally stone dust has some few applications in construction. But using stone dust as an alternative of sand will increase the application of this material in various construction. It can also be used to increase the strength of concrete.

2. METHODOLOGY

According to our objectives the compressive strengths of concrete are observed by replacing fine aggregate with stone dust at different level of replacement namely 10%, 20%, 30%, 40%, 50% and 100%. The experimental program was divided in the following phases

1. Test and determine the physical properties of the materials for project work. The physical properties of materials for work included unit weight, fineness modulus of local Padma sand (PS), Domar sand (DS), Stone dust (SD) and Stone chips (SC) and check out the grade according to BS.
2. Compare grain size distribution percentages with B.S 882 to check out whether the fine aggregate that are prepare by mixing of different percentages of Domar sand, Padma sand and Stone dust are suitable or not.
3. The concrete mix was prepared by taking the proportion of cement, fine aggregate and coarse aggregate as 1:2:4.
4. Water cement ratio was taken as 0.4.

To prepare 78 Nos. of cylinder of size 4 inch diameter and 8 inch height (ASTM C 31) to test for compressive strength after curing of 7 days (39 specimen) and 28 days (39 specimens) taking various percentage of fine aggregate.

The composition of 78 specimens were tabulated below (39 specimens for 7 days curing and another 39 specimens for 28 days curing) :

Table 1. Labeling of specimen of various compositon.

Designation	Cement(C) : Fine aggregate(SD or PS or DS) : Stone chips(SC)
A	Cement: Domar Sand(DS): Stone Chips
B	Cement : 10% SD+90% DS: Stone chips
C	Cement : 20% SD+80% DS: Stone chips
D	Cement : 30% SD+70% DS: Stone chips
E	Cement : 40% SD+60% DS: Stone chips
F	Cement : 50% SD+50% DS: Stone chips
G	Cement: Stone Dust(SD): Stone Chips
H	Cement : Padma sand(PS): Stone chips
I	Cement : 10% SD+90% PS: Stone chips
J	Cement : 20% SD+80% PS: Stone chips
K	Cement : 30% SD+70% PS: Stone chips
L	Cement : 40% SD+60% PS: Stone chips
M	Cement : 50% SD+50% PS: Stone chips

2.1 CASTING AND CURING OF SPECIMEN

The specimens were stripped from moulds after 24 hours of casting. All specimens were carefully cured by immersing in clean water on bath of the laboratory. Most concrete gains strength most rapidly during first week. Structural design is generally based on 28 days strength and about 70% of which is reached at the end of first week after casting. The test specimens were cured 7 days and 28 days in pure tap water.

2.2 SULFUR CAPPING

After curing the specimen were prepared for compressive strength test. But for the uniform load distribution over the bearing area of specimen sulfur capping was done on both bearing surface of each specimen. Molten sulfur compound capping was formed by a vertical capping apparatus as specified in ASTM C 617,and neoprene pad for an unbounded capping system was employed to test the hardened concrete cylinders in accordance with the capping standard's as described in ASTM C1231/C1231M. An elastomeric pad like neoprene pad deforms upon initial loading, conforming to the contour of the cylinder-end, and it is restrained from excessive lateral spreading by its plate and metal ring to provide uniform distribution of the load exerting from the bearing blocks of testing machine to the end of concrete cylinder. In this capping method first sulfur was melted in a heater as shown in the picture. Then the molted sulfur was placed in a metal plate by a spoon. After this specimen was placed on the plate, then keeping the specimen above the plate surface until the molted sulfur hardened. Both side of the specimen was capped by sulfur by following the above procedure.

Figure 1: Sulfur capping.

2.3 COMPRESSIVE STRENGTH TEST

A universal testing machine was used to load the specimen. The test was done by one point loading. At first the specimen was placed in the testing machine as shown in the Figure 3.2 below. Then load was gradually increased from zero until the specimen was cracked or failed. The failure load was noted. After this another specimen was taken and repeats the above procedure to find out the failure load. The strength at this failure load is considered as compressive strength of concrete.

Figure 2: Compressive strength test (Universal testing machine).

Figure 3: Specimens after crushing.

3. ILLUSTRATIONS

3.1 3.1. Stone Dust

Stone dust is a waste material. It is left after screening of stone chips that is produced at the time of crushing stone. It is a granular material. Stone dust has the specific gravity of stone chips. In our experiment, Stone dust is used as fine aggregate (Mahzuz, et al, 2010) and it is passed No. 8 sieve. In this project work stone dust was collected from Dimla, Nilphameri where a large amount of stone is collected and stone chips are produced by crushing. Eventually a huge amount of stone dust produced there.
Following are the properties of stone dust:

1. It is chemically inert.
2. It has good bounding capacity with cement.
3. It may not contain salt, which attracts moisture from atmosphere.
4. It does not contain organic impurities.

Advantages of stone dust are as follows:

1. It may available where construction work is done.
2. It ensures reuse of wastage material.

3. It is cheeper than sand.

Figure 4: Stone dust thrown as waste.

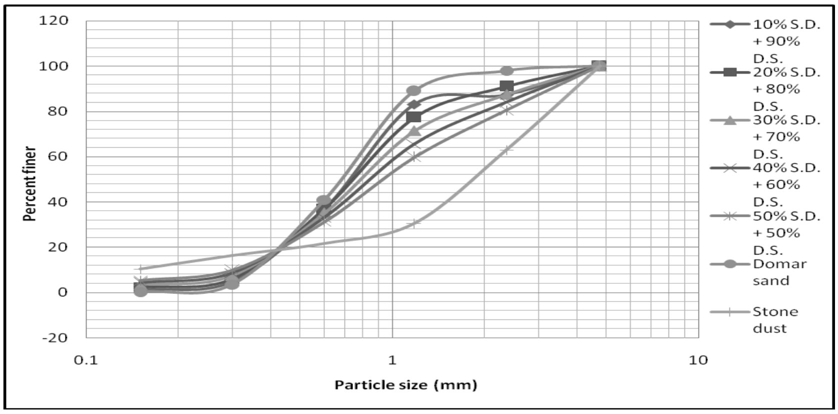

Figure 5: Gradation curve of domar sand, stone dust and mixture of stone dust and domar sand.

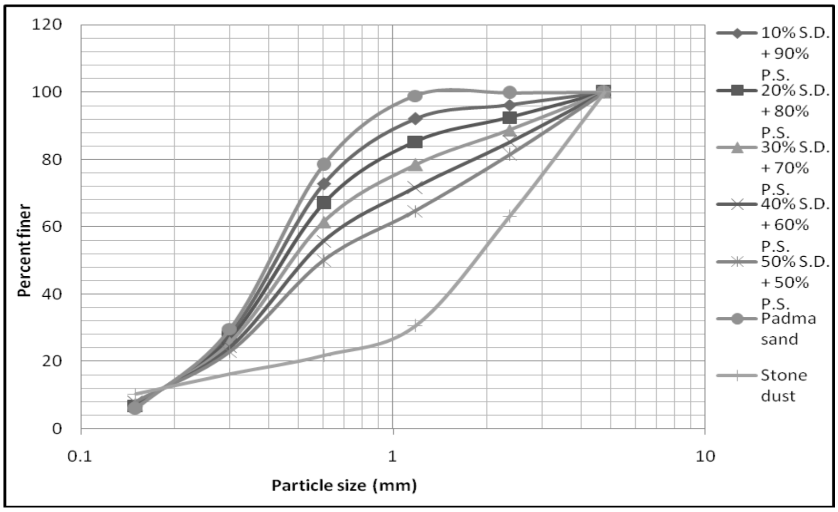

Figure 6: Gradation curve of padma sand, stone dust and mixture of stone dust and padma sand.

In this project work as we use different percentages of stone dust along with domar sand as well as padma sand also simply domar sand, padma sand and stone dust in concrete mix as fine aggregate, it is essential to check the grading of those fine aggregates with standard. The following envelopes show that the grading are reasonable with B.S 882.

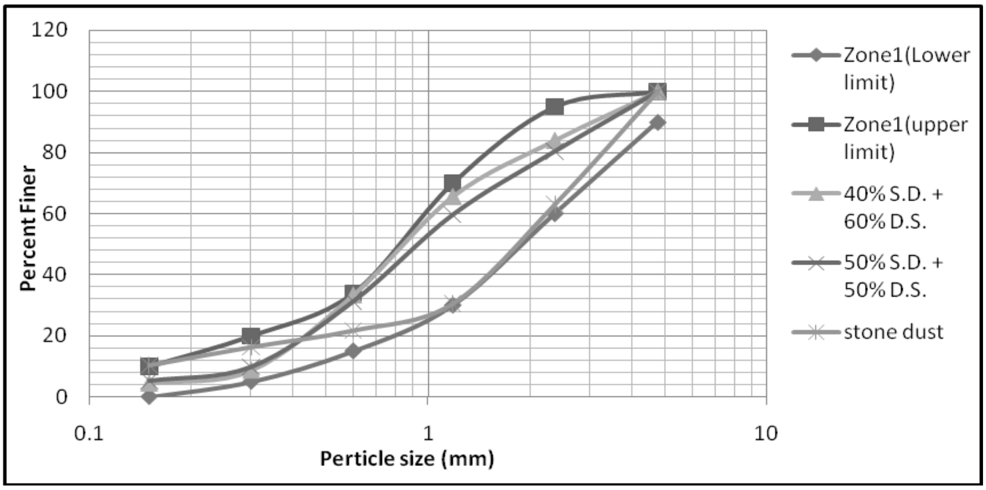

Figure 7: Comparison of grading with B.S. 882, Zone 1.

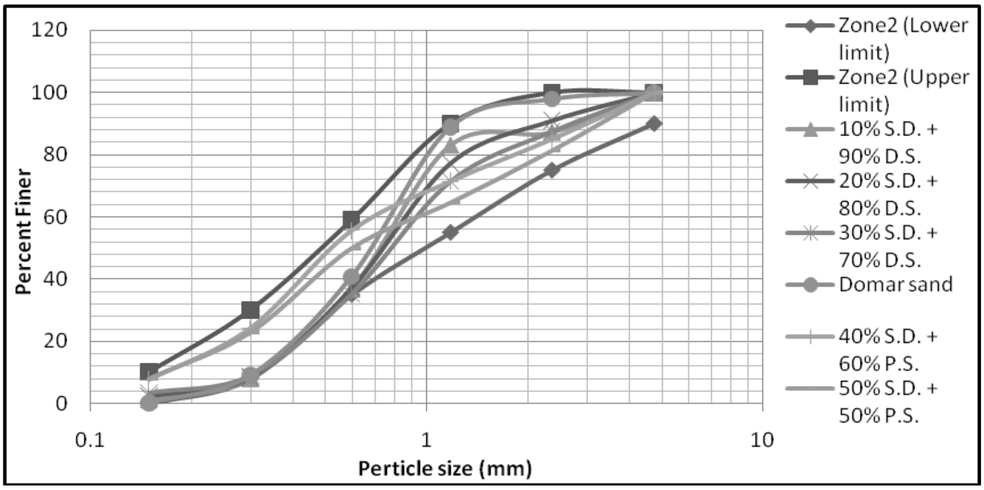

Figure 8: Comparison of grading with B.S. 882, Zone 2.

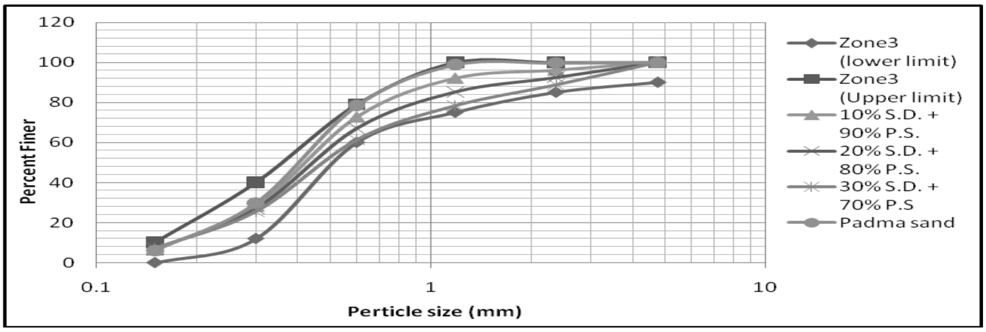

Figure 9: Comparison of grading with B.S. 882, Zone 3.

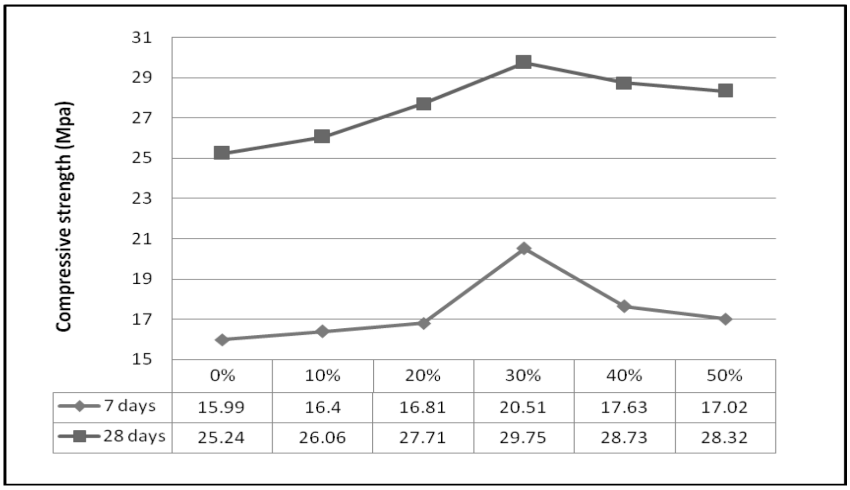

Figure 10: Graphical representation of compressive strength at different curing period when Domar sand is partially replaced with stone dust.

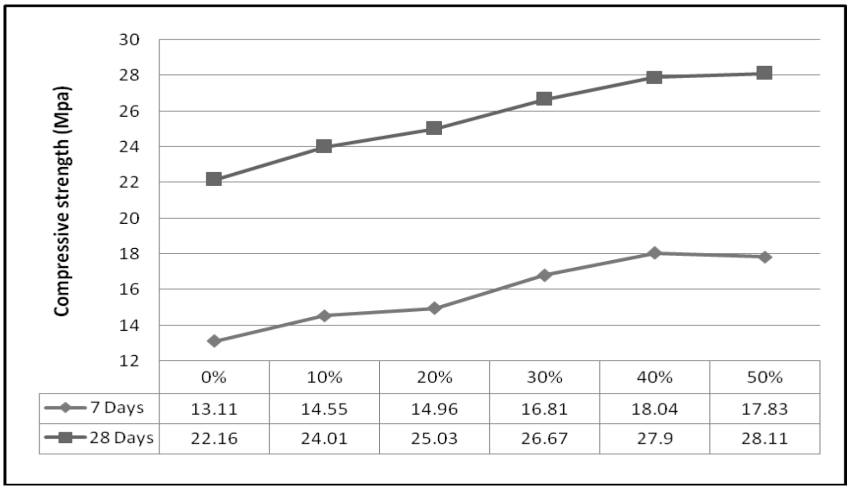

Figure 11: Graphical representation of compressive strength at different curing period when Padma sand is partially replaced with stone dust.

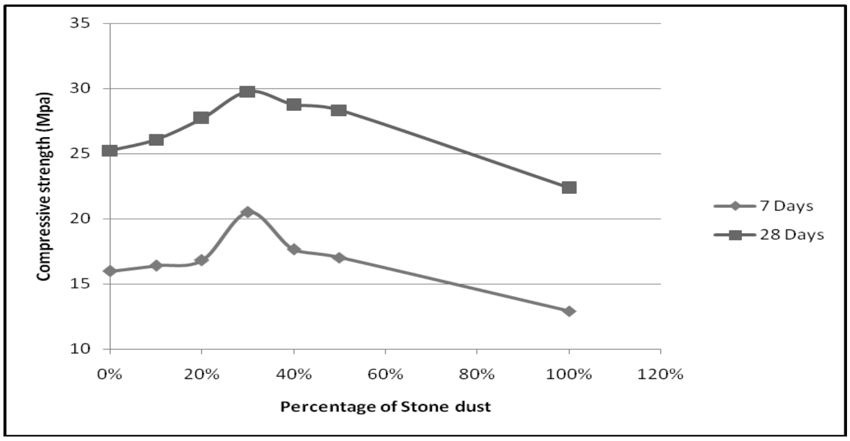

Figure 12: Variation of compressive strength at different curing period when Domar sand is partially replaced with stone dust

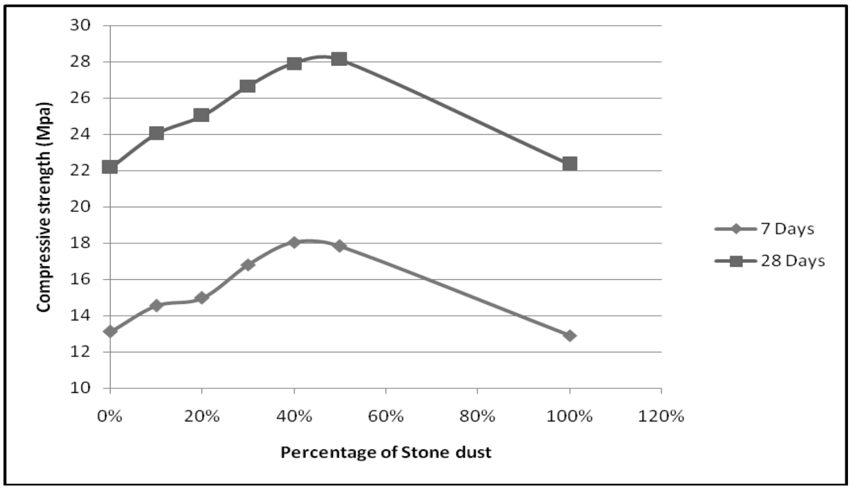

Figure 13: Variation of compressive strength at different curing period when Padma sand is partially replaced with stone dust

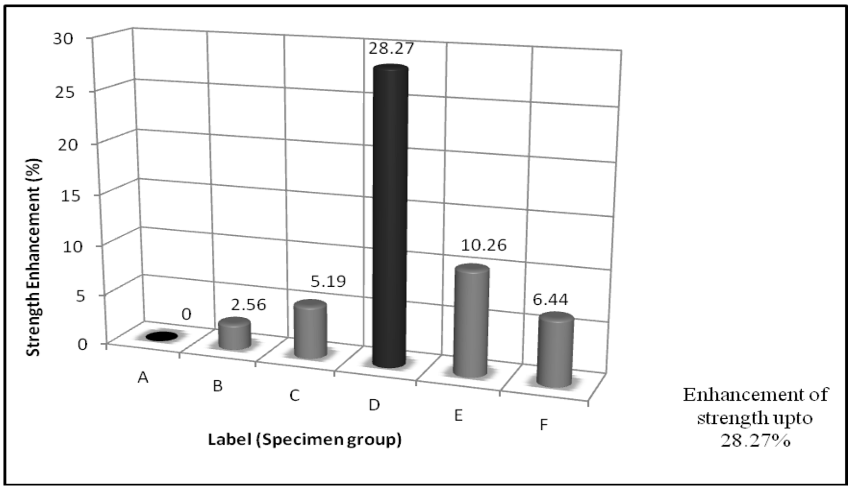

Figure 14: Compressive strength enhancement of the specimen made of mixture of stone dust and domar sand at 7 days curing.

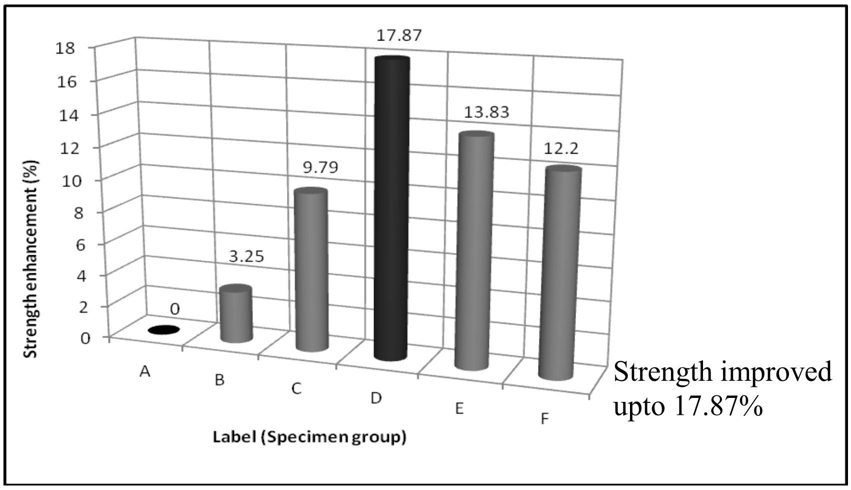

Figure 15: Compressive strength enhancement of the specimen made of mixture of stone dust and domar sand at 28 days curing.

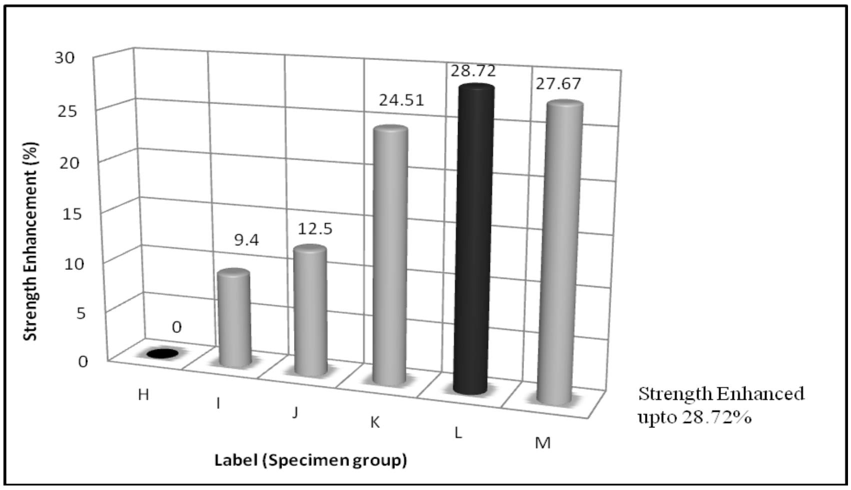

Figure 16: Compressive strength enhancement of the specimen made of stone dust and padma sand at 7 days curing.

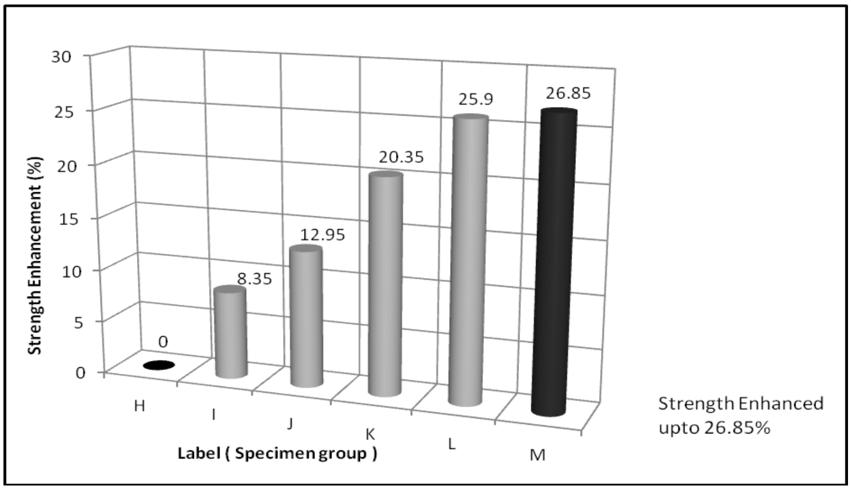

Figure 17: Compressive strength enhancement of the specimen made of stone dust and padma sand at 28 days curing.

4. CONCLUSIONS

Concrete is a heterogeneous mixture of cement, aggregate, and water. Specially, it is an artificial stone having inert materials, binder and water. This study focuses the relative performances of concrete by taking into consideration fine aggregate. The relative compressive strength of concrete made by domar sand as well as local sand (padma sand) have observed here. Same performance was evaluated using stone dust as an partial replacement of natural sand. Based on limited experimental investigation concerning the compressive strength of concrete the following observation is made:

1. Stone dust as waste product from stone crusher left after screening crushed stone. It's used in concrete could save as much as 30% to 50% of natural sand as fine aggregate, while provide the same or much strength.
2. The stone dust satisfies the B.S 882, Zone 1 gradation for not only to partially replace the sand, but for making good concrete.

3. The fine aggregate have got after partially replacing by stone dust satisfies the B.S. 882, Zone 1, Zone 2, Zone 3 gradation for making good concrete.
4. On an overall, the stone dust can be comparable to the natural river sand.
5. The compressive strength of partial replacement of stone dust aggregate concrete is higher than that of the natural sand aggregate concrete at the age of 7 days and 28 days.
6. The compressive strength of concrete in case of domar sand is maximum at 30% partial replacement with stone dust at both 7 days and 28 days curing period. Here we achieved improvement uto 28.27% at 7 days curing and 17.87% at 28 curing with respect to concrete made of domar sand.
7. The compressive strength of concrete in case of padma sand is maximum at 40% replacement by stone dust at 7 days curing and at 50% replacement by stone dust at 28 days curing. Therefore it can be concluded that the compressive strength of concrete is increased upto 40% or 50% replacement. Thereafter it will decrease. Here we get improvement upto 28.72% at 7 days curing and 26.85% at 28 days curing with respect to concrete strength made of padma sand.

This study reveals that in case of concrete the natural sand can be replaced by stone dust. The compressive strength of concrete are not affected by stone dust, rather it provides an improvement to the compressive strength of concrete. This improved concrete can be used where design requires higher compressive strength. By using partially stone dust with padma sand this natural sand can be frequently used in construction, which is rather inferior.

REFERENCES

Ahmed AAM, Yusuf MA (2009). " Using of s tone powder as an alternative of sand" B.Sc. Eng. Thesis, Civil and Environmental Engineering Department, Shahjalal University of Science and Technology, Sylhet, Bangladesh.

Al-Jabri K.S., Hisada M., Al-Oraimi S.K., Al-Saidy A.H., Copper Slag as Sand Replacement for High Performance Concrete. Cement a. Concrete Comp., 31, 7, 483-488 (2009).

Aziz MA (1995). Engineering Materials. Z and Z Computer and Printers, Dhaka, Bangladesh.

Armin Roohbakhshan and Behzad Kalantari, 2012. "Use of Waste Stone Powder in some Civil Engineering Materials and Projects". Department of Civil Engineering, University of Hormozgan, Iran.

Ferguson PM, Breen JE, Jiras JO (1988). Reinforced Concrete Fundamentals, 5th Ed., John Wiley & Sons, p. 1.

H. M. A. Mahzuz, A. A. M. Ahmed and M. A. Yusuf. " Use of stone powder in concrete and mortar as an alternative of sand ". Accepted 6 December, 2010.

Isabelle MC, Dolors V, J ohn DR, Ignac io D, Anabel P (1999). Potential impacts of gravel extraction on Spanish populations of river blennies Salaria fluv iatilis (Pisc es, Blenniidae), Biol. Cons erv., 87(3): 359-367 March, doi:10.1016/S0006-3207(98)00072-X.

Islam MA, Sayem A, Iqbal M, Shahariar MH, Imtiaz S (2010). Prospects for the stone crusher industry in Jaflong region of Sylhet, Proceedings of the conference on Engineering Research, Innovation and Education CERIE, 11- 13 Jan., Sylhet, Bangladesh.

Kameswari KSB, Bhole AG, Paramasivam R (2001). Evaluation of Solidification (S/S) Process for the Disposal of Arsenic Bearing Sludge in Landfill sites. Environ. Eng. Sci., 18: 167-176.

M. S. Shetty, "Concrete Technology", Published by S. Chand & company ltd. New Delhi, India (1997).

Mahzuz HMA, Tajmunnahar HBM (2010). "Assessment of present environmental situation and strategy formulation for future extraction of natural resources of Jaflong" Proceedings of the conference on Engineering Research, Innovation and Education CERIE, 11-13 January, Sylhet, Bangladesh.

Md. Shafi-ul Alam, (2002), "Effect of brick dust as fine particles on the compressive strength of concrete" B.Sc. thesis, Dept. of Civil Engineering, Ruet, Rajshahi.

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Rangwala, "Engineering Materials" Published by House, Anand, India (1969).

Razon Kumar Pal (2006) & Badol Chandro Ray (2006), "Study on strength of concrete by partial replacement of fine aggregate using brick dust". B.Sc. thesis, Dept. of Civil Engineering, Ruet, Rajshahi.

S. KEERTHINARAYANA and R. SRINIVASAN " STUDY ON STRENGTH AND DURABILITY OF CONCRETE BY PARTIAL REPLACEMENT OF FINE AGGREGATE USING CRUSHED SPENT FIRE BRICKS " Universitatea Tehnica, Gheorghe Asachi din Iasi Tomul LVI (LX), Fasc. 2,2010.
http://www.bipcons.ce.tuiasi.ro/content/ArticleInformation.php?Article ID=180

Sharmin R, Ahmed M, Mohiuddin A, Forhat AL (2006). Comparison of Strength Performance of Concrete with Uncrushed or Crushed Coarse aggregate. ARPN J. Eng. Appl. Sci., 1(2): 1-4.

Stone JR, Gilliam JW, Cassel DK,Daniels RB, Nelson LA, Kleiss HJ (1985). "Effect of Erosion and Landscape Position on the Productivity of Piedmont Soils". 677 S. Segoe Rd., Madison, WI 53711 USA, Soil Sci. Soc. Am. J., 49: 987-991.

Sanchez F, Garrabrants AC, Vandecasteele C, Moszkowicz P, Kosson DS (2002). Environmental Assessment of Waste Matrices Contaminated with Arsenic. J. Hazard. Mater., 96: 229-257.

The Star (2009). A weekly publication of the daily star. 18th Sept.

Villalobos S, Lange DA, Roesler JR (2005). Evaluation, testing and comparison between crushed manufactured sand and natural sand technical note. Department of Civil and Environmental Engineering, University of Illinois, 2129 NCEL, MC-250, and Urbana, IL.

Zainab Z. Ismail, Enas A. AL-Hashmi. Reuse of Waste Iron as a Partial Replacement of Sand in Concrete. Waste Manag., 28, 11, 2048-2053 (2008).

STRENGTHENING OF EXISTING RC ROOF SLAB

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ABSTRACT

Different strengthening techniques have been developed so far for the reinforced concrete slabs. The development of these methods was a necessity due to different causes like inadequate maintenance, overloading of the reinforced concrete member, corrosion of the steel reinforcement as well as the available working environment of the project. The existing roof slab of Dr. M. A. Rashid Hall in Khulna University of Engineering and Technology has been suffering from many problems like concrete spalling, reinforcement corrosion, damping etc. which indicates the requirement of an immediate strengthening. In that case ferrocement covering is one of the most effective ways to strengthen the existing roof slab. The major advantages of this system over conventional strengthening techniques are mainly due to the reduction in time consumption and monetary cost as well as the performance of the slab. In this research work three experimental slabs were prepared and strengthened by ferrocement covering. These strengthened slabs were tested in laboratory to find out the overall performance of the slab unit. From the experiment the average deflection was measured 2.48 mm which is within the permissible limit (2.86 mm). And the average failure load indicates that the live load carrying capacity of the strengthened slab is 103 psf which is much greater than the ASTM E 2397-11 suggested value.

Keywords: Roof slab, strengthening technique,ferrocement,ferrocement covering.

1. INTRODUCTION

Strengthening of reinforced concrete (RC) structures is frequently required due to inadequate maintenance, excessive loading, change in use or in code of practice, and/or exposure to adverse environmental conditions. A common feature of a number of different causes of deterioration is that there is a reduction of the alkalinity of the concrete which allows oxidation of the reinforcing steel to take place. This oxidation process leads to cracking of the concrete and possible spalling of the cover to the reinforcement. Several strengthening techniques have been developed in the past and used with some popularity including steel plate bonding, external pre-stressing, section enlargement, and reinforced concrete jacketing. Although these techniques can effectively increase the elements load carrying capacity, they are often susceptible to corrosion damage which results in failure of the strengthening system. Consequently, non-corrosive innovative strengthening systems, such as fibre reinforced polymers, that have the potential for extending service lives of RC structures and reducing maintenance costs, are required to replace old strengthening systems.

Ferrocement can be described as a type of thin composite material made of cement mortar reinforced with wire meshes. The wire meshes are uniformly distributed in continuous layers with relatively small diameters. Romualdi and Iron (1987) were the first to introduce the technique of repair using ferrocement layers. They used the technique for repairing mainly liquid retaining structures such as pools, sewer lines, tunnels, etc. Paramasivam (1987) started investigation concerning the use of ferrocement as strengthening components for the repair and strengthening of reinforced concrete beams. The ferrocement was used, in general, to replace the damaged concrete and reinforcement (if also damaged). The experiment's results shown that the strengthen beams presented improved cracking resistance, flexural stiffness and the ultimate loads compared to the original beams. One of the important conclusions from the experiment was that the improvements depend on the full composite action between the ferrocement layers. The flexural behavior of slabs strengthened with ferrocement was studied by Al–Kubaisy and Zamin (2000). They tested twelve simply supported reinforced concrete slabs under flexural load. The concrete slabs were strengthened with ferrocement in the tension zone cover. In the study the effect of the percentage of wire mesh reinforcement in the ferrocement layer, thickness of the ferrocement layer and the type of connection between the ferrocement layer and reinforced concrete slab on the ultimate flexural load, first crack load, crack width and spacing and load–deflection relationship, have been

taken into consideration. Rifaie and Hassan (1994) investigated the experimental and theoretical structural behavior of thin ferrocement one-way bending elements. The elements of different span and widths were chosen to study their relative feasibility for adoption to roofing of small size residential houses. Nine folded plate models (a channel type cross-section) were constructed and tested to failure. In each of these models, the web-depth, and the thickness of cross-section were kept constant while the span and the flange-width were varied. Alrifaie and Trikha (1988) investigated uniformly loaded ferrocement slabs (500x500 mrn) in which three different arrangements namely a) all layers oriented in one direction: b) all layers oriented in orthogonal direction and c) twin layers-each twin layer consisting of two orthogonally oriented meshes in contact with each other were investigated. A total of 12 square slabs (20 mm and 30 mm thick) were tested under uniform load. Based on the above studies they concluded that the arrangement consisting of twin layers with two meshes orthogonally and placed in contact is superior to the other two arrangements consisting of all the meshes unidirectional oriented or alternate layers equally spaced with orthogonally oriented meshes. The increase in first cracking load is 16% to 24% for 20 mm and 30 mm thick slabs (i.e.,) models. The above increase is 13.2% and 11.75% in the ultimate load value for the 20 mm and 30 mm thick models respectively. Hence, they recommended that for the slabs under bi-axial state of bending the meshes should be arranged in twin layers. Clarke and Sharma (1991) have presented the results on an experimental program taking into account lamination effects. They used 23 square simply supported ferrocement slabs and tested up to failure under biaxial flexure and they found that under bi-axial flexure maximum strength is achieved.KarunkarRao (1998) presented a rational method of computing ultimate flexural strength of a composite unit consisting of precast ferrocement elements composites with reinforced concrete slab units. The development and application of the composite system involving the use of precast ferrocement elements and cast-in-situ concrete slab roofing, basically involved planning of suitable shaped precast ferrocement structural units, testing them individual and in combination with in-situ concrete, to establish and evaluate the composite action. He concluded that the load-deflection data of the structural system under working load conditions give on insight into its structural behavior and that the elimination of the ultimate strength of ferrocement units in flexure with reasonable accuracy develops to a large extent on the stress at first crack and the correct value of a factor K-which accounts for the mobilization of a tone in the flexure span. He has also suggested further studies to establish design norms.
This study presents preliminary investigations of structural behavior of existing RC roof slab and the overall performance of strengthened slab by ferrocement covering.

2. METHODOLOGY

2.1 Investigation on Existing RC Roof Slab Condition of Dr. M. A. Rashid Hall

During the course of this research work, a preliminary investigation on existing roof slab condition of Dr. M. A. Rashid Hall was carried out to determine whether it is necessary to strengthen or not.

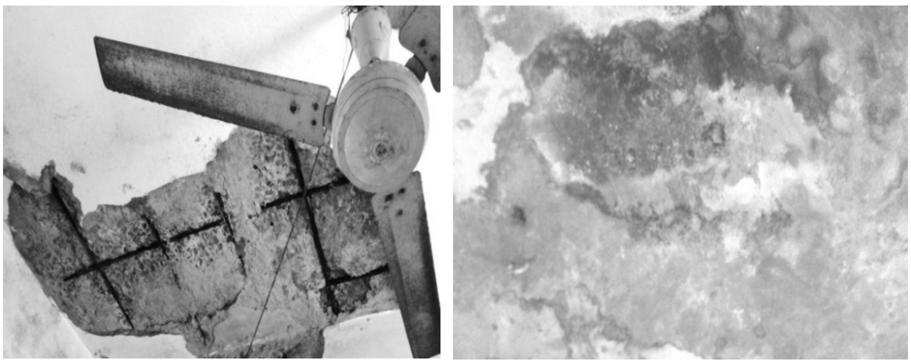

Figure 1: Existing Roof Slab Condition of Dr. M. A. Rashid Hall

The above figuredepicts concrete spalling, reinforcement corrosion and damping which occurred on the existing roof slab of Dr. M. A. Rashid Hall.The preliminary investigation indicates the requirement of an immediate strengthening of existing RC roof slab of Dr. M.A. Rashid Hall.

2.2 Strengthening Details

The presented technique has the potential of increasing the structural capacity of the structural members or in case of damaged slabs, to restore the original capacity of the section.

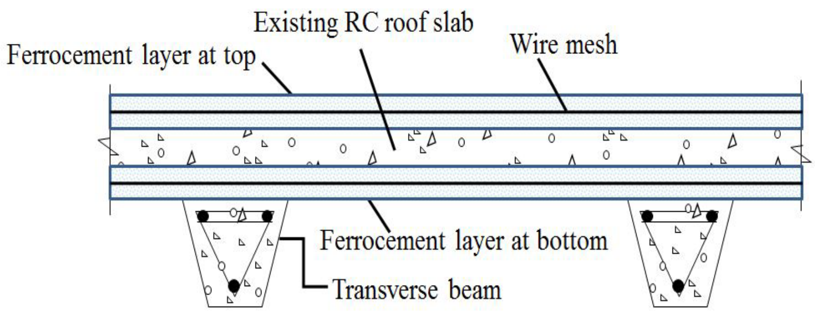

Figure 1: Longitudinal Section of Strengthened Slab

According to the present strengthening technique, at first the lime course of the existing roof slab is to be cut out. Then ferrocement layer of about one inch is to be introduced at top and bottom of the existing slab. Finally transverse beam of suitable section is to be provided under the strengthened slab at a regular interval which is to be connected to the existing longitudinal beams with steel bracket.
The following figure is the sectional plan view of 2nd floor of Dr. M. A. Rashid Hall on which the preliminary investigation was carried out and strengthening work is recommended.

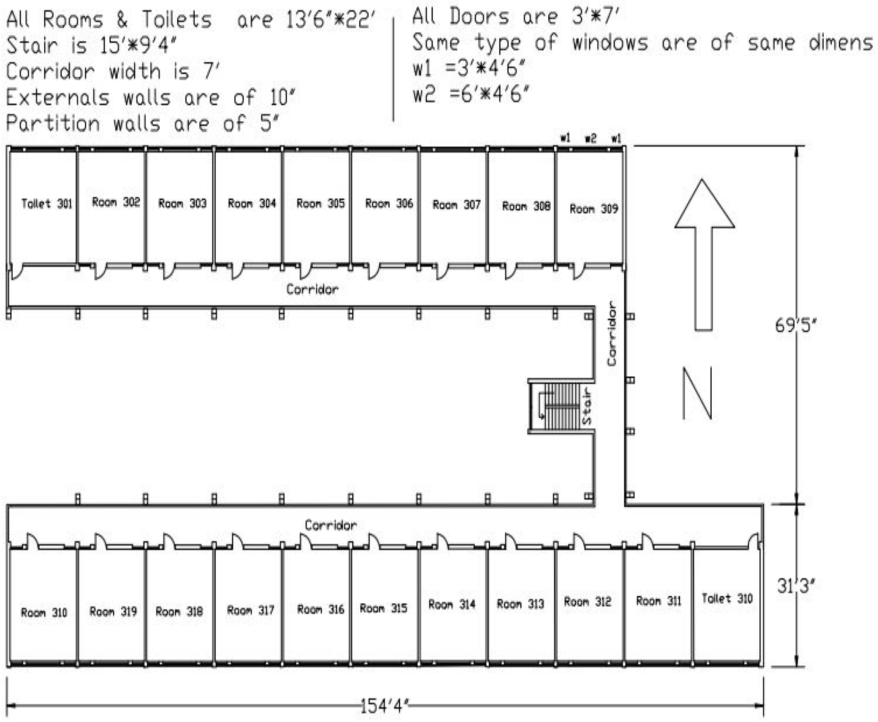

Figure 3: Plan of Dr. M. A. Rashid Hall (2nd floor)

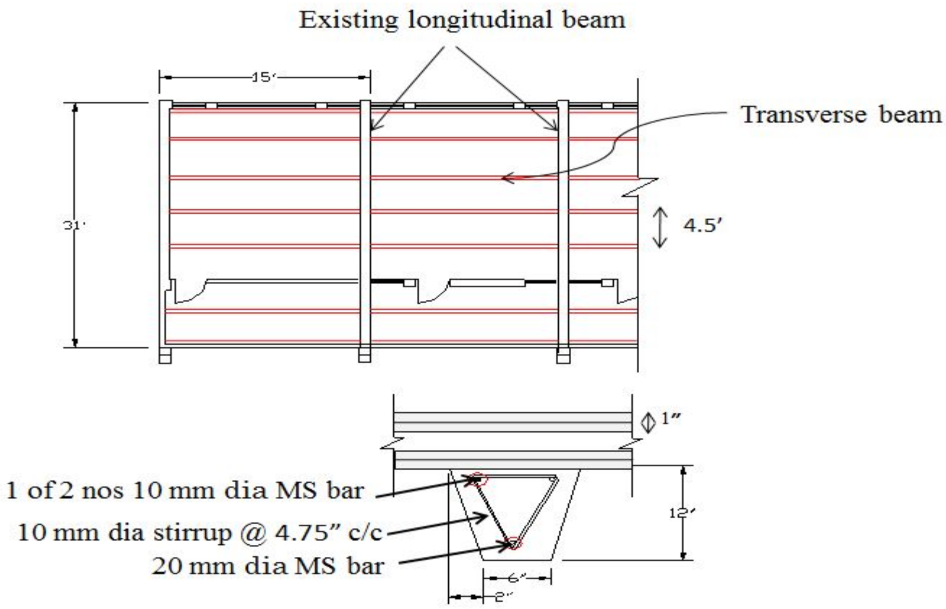

Figure 2: Cross Section of a Transverse Beam

Figure4 illustrates the transverse beam positioning and reinforcement detailing. In each room, consecutive transverse beam is to be provided at a distance of 4.5 feet center to center. In the present study trapezoidal section for the transverse beam is suggested in which 2 nos of 10 mm dia MS bar at top and 1 nos of 20 mm dia MS bar at bottom are to be provided.

2.3 Test of the Materials

2.3.1 Fineness Modulus of the Sand

In the present work Sylhet sand was used as fine aggregate for the casting of the slab specimens. The F.M. value was determined by using no. 100, no. 50, no. 30 no. 16, no. 8, and no. 4 sieves according to ASTM C33. For this purpose, the sand was dried to a constant temperature of 110^0C for 24 hours in an oven and then 1200gm sample was taken for the determination of F.M. value.

2.3.2 Compressive Strength Test of Mortar

Mortar was prepared from normal Portland cement and Sylhet Sand (FM=2.96), mixed in ratio of 1:2 by weight. For preparation of workable paste, the amount of water required was determined from flow test of mortar as proposed by ASTM Standard requirement of specification C 150.

Figure 5: Compressive Strength Test of Mortar

2.4 Construction of the Specimens

In this research work, three numbers of slab specimens were constructed providing single layer of GI wire mesh at both top and bottom. During the construction of specimens skilled labor and requisite quality control were achieved.

2.4.1 Specification of the Slab Specimens

Slab size: 4.5′x1.75′x5″
Reinforcement type: GI Wire mesh (12.7 mm opening)
Reinforcement position: Top and Bottom
Thickness of ferrocement layer: 1″ for each top and bottom

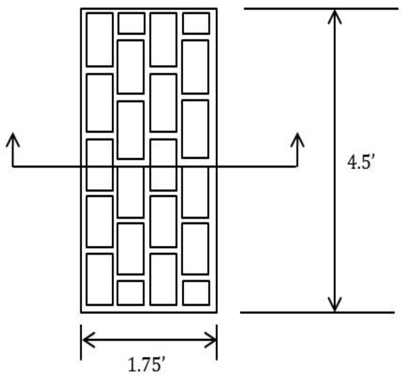

Figure 6: Longitudinal Section of Slab Specimen

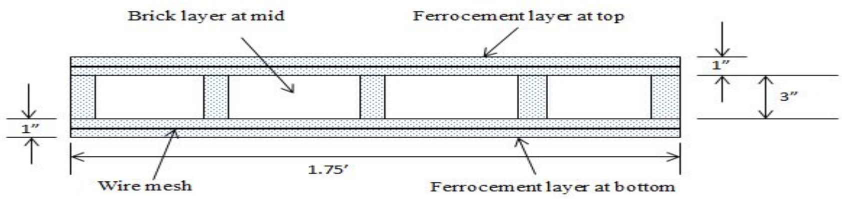

Figure 7: Cross section of Slab Specimen

2.4.2 Casting of the Specimens

Mortar mixing was done for the casting of slab specimens. For mortar mixing, sylhet sand (F.M.=2.96) and Ordinary Portland Cement was used and the cement-sand ratio was kept 1:2. During mortar mixing the water-cement ratio was maintained 0.50.Due to the limitation of testing the roof slab in existing condition, three numbers of identical slab specimens representing the mid-section of the existing slab were constructed using bricks as replacing material of concrete.Then, for strenthening purpose of the slab single layer of ferrocement covering was provided at top and bottom.

Figure 8: Casting of the Slab Specimen

2.5 Experimental Programme

The test was carried out on the slab specimens using hydraulic jack, load cell, DATA-LOGGER (TDS 303) and other accessories to determine the behavior of strengthened slab specimens.

2.5.1 Experimental Set up

The schematic diagram of experimental test set up for testing the slab specimens is shown below. During the test of the slab specimens two point loading was applied.

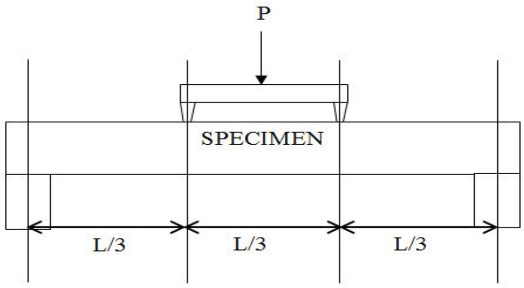

Figure 9: Schematic Diagram of Test Set Up

2.5.2 Testing of the Slab Specimens

The following steps are the testing procedure which was carried out during the test of the slab specimens.

- The slab specimens were placed horizontally on the simply supported condition as shown in Figure 10.
- A 10 ton capacity hydraulic jack along with a load cell mounted vertically on a steel plate between the bamboo and the slab specimen.
- After that LVDT were placed at the mid-point and at the one third point of the slab specimen.
- Then the incremental vertical load was applied and deflection measurements against the applied load at two LVDT points were simultaneously collected by means of DATA-LOGGER (TDS 303).

Figure 10: Laboratory Test set up of Slab Specimens

3. RESULTS AND DISCUSSION

3.1 Material Properties

Table 1: Fineness Modulus of Sand

Sieve no. (ASTM)	Weight retained (gm)	Cumulative weight retained (gm)	Cumulative percent weight retained	Fineness Modulus(F.M)
No. 4	0.0	0.0	0.0	2.96
No. 8	89.7	89.7	7.5	
No. 16	326.2	415.9	34.7	
No. 30	369.4	785.3	65.4	
No. 50	278.8	1064.1	88.7	
No. 100	130.1	1194.2	99.5	
Total	1194.2		295.8	

Table 2: Tests Results for Compressive Strength of Cement Mortar (cube)

Age (days)	Ratio (1:2)	Area (in^2)	Applied Load (lb)	Compressive Strength (psi)	Average Compressive Strength (psi)
3	Cement: Sylhet Sand	4	14800	3700	3850
			16000	4000	
			15400	3850	
7	Cement: Sylhet Sand	4	15800	3950	4110
			16500	4125	
			17000	4250	

3.2 Failure Pattern of the Slab Specimens

Slab specimens strengthened with GI wire mesh fails with warning like as ductile behavior. During the failure of slab, the upper portion of slab is governed in compression and lower portion of that slab is governed in tension. Due to applied vertical load, flexure failure occurred at the mid-section of each slab specimens. Failure pattern of slab specimen is shown below.

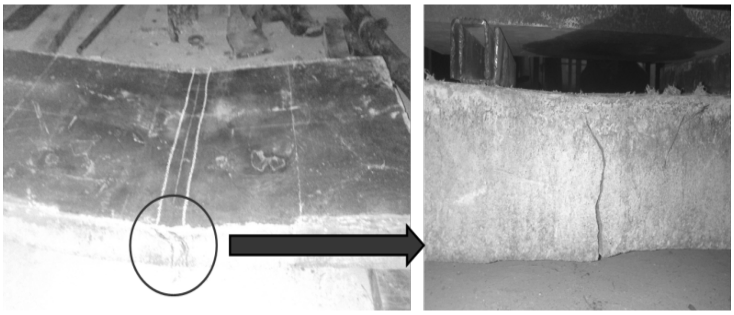

Figure 11: Failure Pattern in Slab Specimens

3.2 Graphical Representation of the Test Results for Slab Specimens

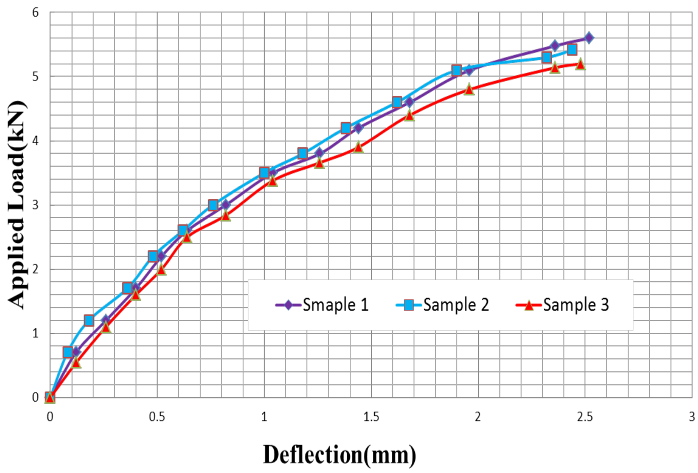

Figure 12: Applied Load vs. Deflection Curve

Table 3: Test Results of the Slab Specimens

Sample Designation	Failure Load (kN)	Average Failure Load (kN)	Deflection (mm)	Average Deflection (mm)
Sample 1	5.60	5.41	2.52	2.48
Sample 2	5.42		2.44	
Sample 3	5.20		2.48	

Length of the specimen= 4.5′
Allowable deflection,

$$\frac{l}{480} = \frac{4.5 \times 12}{480} = 0.1125'' = 2.86\ mm$$

Average deflection from Table7.4 = 2.48 mm

So, from the test results it is clear that the average deflection of the strengthened slab specimen is within the allowable limit.

From Table 7.4, the average failure load,

$$5.41\ kN = \frac{5.41 \times 1000 \times 2.2}{9.81} = 1213.25\ lb$$

Slab dimension = 4.5'x1.75'

Load carrying capacity of the strengthened slab per square area,

$$\frac{1213.25}{4.5 \times 1.75} = 154\ psf$$

Dead load of the slab specimen,

$$\frac{2}{12} \times 125 + \frac{3}{12} \times 120 = 51\ psf$$

Live load carrying capacity of the strengthened slab,

$$154 - 51 = 103\ psf$$

In roof slab ASTM E 2397-11 suggested value of live load is 40 psf.

So, here the live load carrying capacity of the strengthened slab is much greater than the ASTM suggested value.

4. CONCLUSIONS

Analyzing the test results following conclusions can be drawn:

1. Existing RC roof slab condition of Dr. M A Rashid Hall indicates that it should be strengthened.
2. The average deflection of the strengthened slab specimens is 2.48 mm which is within the allowable limit (2.86 mm).
3. The average live load carrying capacity of strengthened slab is 103 psf which is much greater than the value (40 psf) suggested by ASTM E 2397-11.

Finally, the observations of measured values of deflections and load carrying capacity indicate that, the ferrocement covering can be used in strengthening the existing RC roof slab.

REFERENCES

Al-Kubaisy M.A., Zamin J.M. (2000), "Flexural Behaviour of Reinforced Concrete Slabs with Ferrocement",Construction and Building Material, 14, 245–252.

Al-Rifaei, Wail N. and Hassan, Arsalan H (1994), "Structural Behavior of Thin Ferrocement One-way Bending Elements", Journal of Ferrocement, 24(2), pp.115-126.

Al-Rifaie, W.N.andTrikha, D.N. (1988), "Effect of Arrangement and Orientation of Hexagonal Mesh of the Behaviour of Tow-Way Ferrocement Slabs". Proceedings of the Third International Symposium on Ferrocement, 1988, India, Journal of Ferrocement, Volume 20, No.3.

ASTM C109 (Jan. 10, 1999), "Standard Test Method for Compressive strength of HydraulicCement Mortar", American Society for Testing and Material.

ASTM E2397 (2011), "Standard Practice for Determination of Dead Loads and Live Loads Associated with Vegetative (Green) Roof System", American Society for Testing and Material.

KarunkarRao. (April 1998), "Flexural Design of Ferrocement: Computerized Evaluation and Design Aids" Journal of Ferrocement, pp101-117.

Paramasivam P. (1987), "Laminated Ferrocement for Better Repairment", Concrete International : Design and Construction, 9, 34–38.

R. P. Clarke, A. K. Sharma (1991), "The Experimental Behaviour of FerrocementFlat Plate under Biaxial Flexure," Journal of Ferrocement, pp.127-136,.

Romualdi, J. P. (1987), "Ferrocement for Infrastructure Rehabilitation", Concrete International: Design and Construction, vol.9, no. 9, pp. 24-28.

COMPARATIVE STUDY ON RAPID CHLORIDE DURABILITY TESTS ON SUPPLEMENTARY CONCRETES

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ABSTRACT

An experimental investigation have been conducted to study the comparative chloride durability measured by two recently developed electro-chemical tests on concrete prepared with Portland cement and additions consisting pozzolanic (Fly ash) and latent hydraulic (Ground Granulated Blastfurnace Slag - GGBS) mixes and also with limestone powder. Rapid chloride migration tests considered for this study are 1) Potential Difference (PD) test and 2) Multi-Regime (MR) test. Both the tests measure chloride durability in terms of D, the Coefficient of Chloride Diffusion. The PD and MR test results showed that in the early ages, 100% Portland cement concrete performed well against chloride diffusion. However, fly ash and GGBS concrete showed higher resistance against chloride migration at later stage. At equal strength grade, w/c ratio and age, GGBS concrete had the highest resistance against chloride among other cement types.

Keywords: Concrete, Rapid Chloride Migration, Supplementary cements, two-cell test

1. INTRODUCTION

Chloride is one of the most deleterious factors for concrete with embedded steel – reinforced concrete as known. Though reinforced concrete, over the decades, has been serving as most popular construction material for its strength and durability, the corrosion due to chloride impose a huge threat by spalling of surface, reduction in cross-sectional area, disintegration and total structural failure. Concrete structures that have been built in corrosive environment start deterioration well before their design life end (Hardjito *et al.* 2004). Coastal and marine concrete structures are highly exposed to chloride intrusion. Highway structures are also subjected to chloride attack where de-icing salt is applied in cold weather countries. Recent developments in the research of supplementary cementitious materials have already established the standards for structural performance; however, researchers have been continuing there investigation for the optimum durability performance of supplementary cementitious materials. This paper comparatively studied the chloride durability of supplementary cementitious materials by rapid chloride migration methods.

1.1 Supplementary concretes

Although modern Portland cements have much improved properties, its production is highly energy-intensive and responsible for significant amount of carbon-dioxide gas emission. Global yearly production of Portland cement accounts for 7% carbon-dioxide gas release into the atmosphere while each ton of production requires 4GJ of energy (Mehta 2001). It has already been established that utilization of supplementary cementitious materials, which are by-products of other processes, has sustainably improved the cement and concrete industries by reducing environmental impacts associated with cement production (Papadakis 2000). Fly ash and Ground Granulated Blust-furnace Slag (GGBS) are the examples of such materials produced by thermal power plants and metallurgical industries respectively.

Addition of fly ash and GGBS has been reportedly popular means of resisting chloride ingress into concrete among majority in the researchers' community. Inclusion of these cements tends to reduce chloride permeability and risk of corrosion in turn. The improvement has been associated with a number of physical and chemical properties of these materials, resulting in changes in both pore chemistry and microstructure. Pozzolanic properties of these supplementary cementitious materials impart dense microstructure into concrete which is beneficial against chloride ion ingress (Jain and Neithalath 2010). In addition to this, nature of the capillary pore system, secondary hydration reaction and chloride binding capacity of pozzolanic materials significantly increase the chloride resistivity of concrete (Castellote *et al.* 1999).

1.2 Rapid Chloride Migration

Based on the mechanism of natural chloride diffusion and rapid electrochemical migration of chloride ions a number of test methods have been developed in order to measure chloride diffusion into concrete specimens. In practice, all these tests measure the coefficient of diffusion, *D* or equivalent, which is often used as a fundamental parameter to describe the ease with which chloride is transported through concrete. A number of natural diffusion test have been proposed by several researchers. Concentration difference method (CD) was proposed by Dhir (Dhir *et al.* 1990) that determines the diffusion of chloride in the steady-state condition. Andrade (Andrade *et al.* 1994) proposed a ponding test as a comparative test to compare diffusivity value for accelerated migration test. This test takes one year to complete. Nordtest NT BUILD 443 was proposed as part of the 'chlortest research project' to evaluate the resistance of concrete against chloride penetration. It is based on a bulk diffusion test running for about 90 days with 30 days preconditioning. Since testing of 'natural' chloride penetration is time consuming, attempts have been made to calculate the coefficient of diffusion, D by accelerating the rate of penetration of chloride ions with the application of electrical field (Andrade 1993). Since 1980s, rapid test for chloride diffusion has been made possible by the standardisation of a test method by Whiting (Whiting 1981) in the ASTM. Several researchers proposed what can be called as migration type tests based on the two cell test principles.

Among others who proposed similar test methods are Dhir *et al.* (Dhir *et al.* 1990), Tang and Nilsson (Tang and Nilsson 1992), Andrade et al. (Andrade 1993; Andrade *et al.* 1994; Castellote *et al.* 2001) and others (Truc *et al.* 2000; Friedmann *et al.* 2004; Stanish *et al.* 2004). The test methods adopted in this study are PD (Potential Difference) test and MR (Multi-Regime) Test. Schematic diagrams of these test are shown in Figure 1 and 2.

2. METHODOLOGY

The experimental programme considered comparative study on different cement types, therefore alternative cements including Fly Ash, GGBS and limestone filler were introduced as partial replacement of Portland cement. The chemical and physical properties of Portland cement and other supplementary cementitious materials as determined by quantitative X-ray diffraction technology are provided in Table 1. Only major Oxides are shown. The mix proportions were adopted from the basis of the minimum requirement from the BS 8500-1 (British Standards Institution 2006) for XD3 class which is stated as the most severe chloride exposure class with cyclic wet and dry environmental condition. Mix proportion is shown in Table 2.

The mixing was carried out in accordance with British Standard BS 1881-125: 1986 in a horizontal pan type mixer. Properties of fresh and hardened concrete i.e. slump and compressive strength are shown in Table 3. Workability of concrete mixes was measured in terms of slump in accordance with the description provided in BS EN 12350-2: 2009. General trend of strength development for all cement types are similar, i.e. compressive strength increased with time. Decrease in w/c ratio also showed increase in strength as usual. However, trends of strength development of Fly Ash and GGBS concrete suggest that these concrete types will gain similar or higher strength at their later age due to slow and continued hydration.

2.1 Potential Difference (PD) test method

PD test was conducted on 25 mm thick specimens cured for 7 days prior to test. Each specimen was sealed under a diffusion cell containing 800 ml. of deionised water. The cell was then put into immersion tank containing 5-Molar NaCl solution. A graphite rod inserted into diffusion cell and a steel sheet at the bottom of immersion tank was connected to power source, maintaining a potential difference of 7.5V. Fig 1 shows details. Chloride concentration was determined by computerised titration method by using nitric acid and silver nitrate. Only 0.5 ml of sample from diffusion cell at 12 hours interval was required for titration. The test runs for 7-14 days. PD index (I) was calculated from the modified Fick's First Law of diffusion as proposed by Dhir et al. (1990) which is

$$\ln(C_1 - C_2) = \frac{IA}{Vl}(t_n - t_0) + \ln C_1 \quad (1)$$

Where, C_1 = chloride concentration in immersion tank, ppm; C_2 = chloride concentration in cell reservoir, ppm; I = PD Index, cm^2/s; A = transmission area of the concrete test specimen, cm^2; V = volume of the cell reservoir (litre); ℓ = thickness of test specimen, mm; t_0 = time corresponding to the projection of the abscissa at $C_2 = 0$; t_n = time at conclusion of experiment with concentration in cell of (C_2).

Table 2: Chemical composition of cements determined by X-ray Diffraction

Major Oxides %	**Cement and addition types**			
	PC	Fly ash	GGBS	Limestone
CaO	65.17	3.560	40.18	81.55
SiO_2	20.68	45.90	35.02	0.110
Al_2O_3	5.170	21.05	11.83	0.060
Fe_2O_3	2.970	10.05	0.430	0.020
MgO	1.020	1.400	7.270	0.170
MnO	0.050	0.080	0.530	0.030
TiO_2	0.490	1.120	0.670	-
K_2O	0.580	2.820	0.570	0.010
Na_2O	0.300	1.270	0.310	0.070
P_2O_5	0.220	0.450	0.020	0.010
Cl	0.030	0.000	0.000	0.000
SO_3	3.010	1.260	1.030	0.030

Table 3: Constituents proportion adopted from BS-8500

Proportion & Designation	**Constituents Material (kg/m^3)**									
	w/c ratio	**Cements**						**Aggregates**		
		PC	**GGBS**	**Fly Ash**	**Limestone**	**Total Cement**		**Fine**	**10/4 mm**	**20/10 mm**
100% PC M1	0.35	380				380		736	403	748
	0.40	380				380		729	399	740
	0.45	360				360		732	401	745
70% PC 30% FA M2	0.35	266		114		380		716	392	729
	0.45	252		108		360		713	390	725
	0.55	224		96	10	330		707	396	735
30% PC 70% GGBS M3	0.40	114	266			380		725	397	736
	0.50	102	238			340		733	401	746
	0.55	96	224		10	330		723	404	751
85% PC 15% Limestone M4	0.35	323			57	380		736	403	748
	0.40	323			57	380		729	399	740
	0.45	306			54	360		732	401	745

Table 4: Slump values and strength development

Proportion & Designation	**Concrete properties**				
	w/c ratio	**Achieved Slump (mm)**	**Compressive Strength (N/mm^2)**		
			7 Days	**28 Days**	**90 Days**
100% PC M1	0.35	170	53.7	72.4	84.8
	0.40	131	46.2	69.3	78.3
	0.45	210	39.4	54.3	63.2
70% PC 30% FA M2	0.35	84	28.7	52.1	67.7
	0.45	185	26.5	40.8	49.2
	0.55	98	19.8	26.7	39.3
30% PC 70% GGBS M3	0.40	60	27.4	49.6	70.4
	0.50	140	28.4	50.4	66.1
	0.55	160	13.9	27.9	40.3
85% PC 15% Limestone M4	0.35	120	64.5	81.4	89.4
	0.40	60	57.3	66.3	79.9
	0.45	80	41.9	55.3	59.1

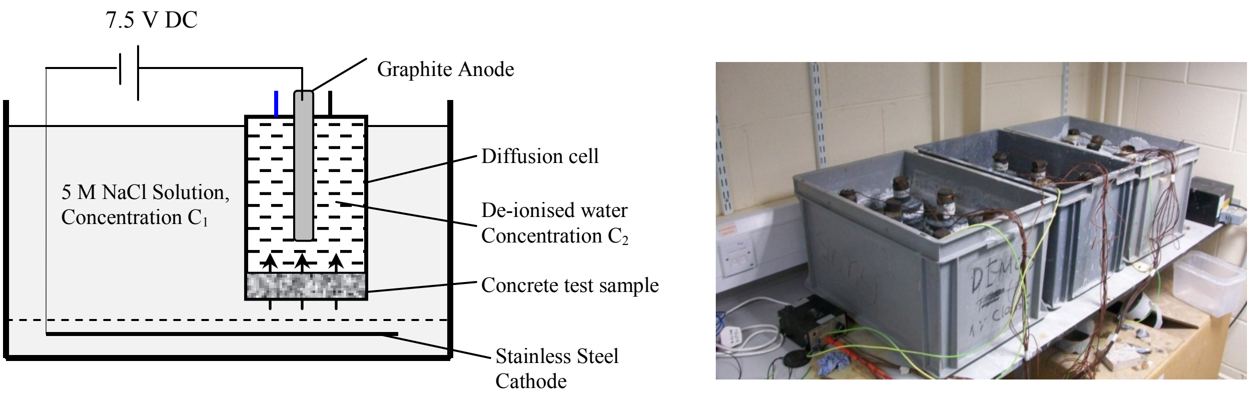

Figure 3: Schematic diagram and test setup of PD test

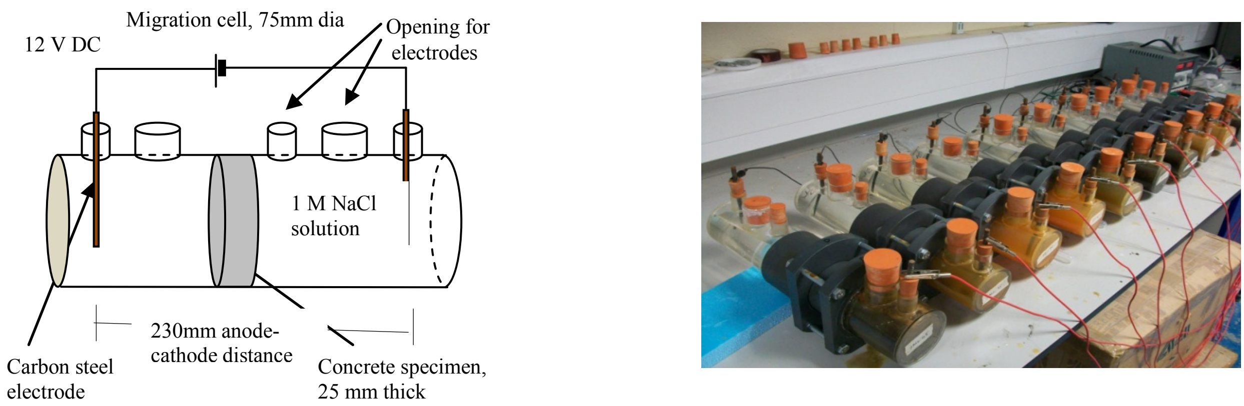

Figure 4: Schematic diagram and test setup of MR test

Table 5: Comparative features of PD and MR test

	PD Test	**Multi-Regime Test**
Concrete specimen	100mm Ø, 25mm thick	75mm Ø, 25mm thick
Potential	7.5V dc	12V dc
Orientation	Vertical	Horizontal
Catholyte Type	5 Molar NaCl, in tank	1 Molar NaCl, in cell
Anolyte volume & type	800ml, Distilled water	500ml, Distilled water
Anode type	38mm Ø carbon (inert)	6mm Ø deformed steel bar (corroding
Cathode type	Stainless steel sheet	6mm Ø deformed steel bar (corroding)
By-product	FeCl	Chlorine gas
Measurement	Chloride content titration	Conductivity
Measurement	Chloride content titration	Conductivity
Anode-cathode distance	125mm	230mm
Chloride migration	Vertical	Horizonal
Equation	Modified solution to Fick's First law	Modified Nernst-Planck

2.2 Multi-regime (MR) test method

Multi-regime chloride test was carried out as described and developed by Castellote *et al.*(Castellote *et al.* 2001) As shown on Fig 2, this test consists of two compartments containing 1-Molar chloride solution and deionised water in upstream and downstream cells respectively. When an external potential of 12V was applied, the cathode placed in chloride solution drives the chloride ions to the downstream cell through the specimen where the anode is placed. Determination of the amount chloride concentration is based on measuring the conductivity of the anolyte solution of the downstream cell instead of analysing it. Steady-state (linear relationship between Chloride concentration and time) diffusion coefficient, D_s is calculated from the Modified Nernst-Planck equation, whereas non-steady state diffusion coefficient can also be calculated by considering the time taken by chloride ions to achieve a constant flux. Steady state diffusion coefficient, D_s was calculated from the Modified Nernst-Planck equation

1.1.1.1.

$$D_s = \frac{JRTl}{zFC_1\gamma\Delta\phi} \tag{2}$$

1.1.1.2. Where,

1.1.1.3.

$$I = \frac{(mmol_{ssf} - mmol_{ssi})\times 10^{-3}}{St_{ss(f-i)}} \tag{3}$$

Here, J = Flux of chlorides during steady state period, mol/cm^{2}s; S = effective surface area of the test specimen, cm^2; t = duration of steady state, sec; C_1 = Cl concentration in the catholyte , mol/cm^3; γ = activity coefficient of catholyte solution; $\Delta\Phi$ = average effective voltage across specimen, volts ;l = specimen thickness, cm; R = perfect gas constant, cal/mol K ;T = average temperature, °K; z = ion valence, for chloride ; F = Faraday's constant, cal/V

2.3 Difference between PD and MR Test method

Usually results obtained from PD test by using Fick's First Law of diffusion were different than diffusion coefficient resulted from MR tests using Nernst Planck equation. These differences in the results were not only due to use of different equation but also due to the difference in physical and methodological characteristics of these test methods. Differences in salient features of these tests are presented in Table 4.

3. RESULTS AND DISCUSSION

Although duly designed concrete mix satisfies strength requirements, it may not always be durable against environmental exposures. It is generally assumed that high strength concrete is better in durability. Basis of this assumption may be drawn from the fact that microstructure of concrete influences both its strength and durability. Therefore, a relationship between strength and durability of concrete can be expected. Initially, concrete with 100% Portland cement showed better strength in compressive test than that of concrete with fly ash and GGBS. However, these supplementary materials gained improved strength at later stage. This was due to secondary pozzolanic hydration. This paper focuses its discussion mainly on chloride durability aspect of concrete samples.

3.1 Chloride migration

Like most of the two-cell test methods, the PD and MR tests also use traditional two-cell arrangement, where one compartment contains chloride solution and the other filled with distilled water. Although the basic principle of these tests is same, their geometrical arrangement, analysis method etc are different, which result in difference in their measured diffusion indices. Results of rapid chloride migration tests are compared across the effect of w/c ratio, cement types, strength on chloride diffusion. The PD and MR test were conducted on 28 and 90 days old specimen. PD index and Diffusion index were calculated from the equation 1, 2 and 3.

3.2 Effect of w/c ratio on chloride diffusion

It is a well known fact that chloride diffusion decreases with the reduction in water to cement ratio due to improved pore structure (Song *et al.* 2008). Effects of w/c ratio on chloride diffusion (PD index) with different cements are shown in Figure 3 and 4. For all of the mixes it is evident that chloride diffusion rate changes with the change in w/c ratio.

For Portland cement concrete (M1), the changes in PD index for 28 days old concrete with w/c=0.45 was more than 2.5 times greater than the PD index with w/c=0.35 (Figure 3.1). At 90 days, the increases in PD index across these two w/c ratios were roughly similar. Significant changes with varying w/c ratio were also evident in case of Portland-Fly ash (M2) and Blastfurnace (M3) concrete. For both of the cement types, the PD index varied nearly 1.5 times between two immediate w/c ratios at all ages. A different trend was noticed in the case of Portland-limestone (M4) concrete. Although, increase in PD index with increasing w/c was evident, the increase was not as substantial as that of other concrete types. On average, order of 1.2 times increments was evident for two consecutive w/c values at all ages.

A similar tendency of diffusion coefficient with altering w/c ratio was noticed when the specimens were tested by Multi-regime method. Figure 5 and 6 shows the *D* values at varying w/c ratio for 28 and 90 days old specimens. The diffusion coefficients were distinctly rising with the w/c values.

3.3 Effect of cement types on chloride diffusion

Other than w/c ratio, age and concrete grade of different cement types showed different trend of chloride diffusion when tested with rapid migration tests. It has been reported by several researchers (Hornain *et al.* 1995; Dhir *et al.* 1996; Luo *et al.* 2003; Sharfuddin Ahmed *et al.* 2008; Yuan *et al.* 2009) that fly ash, GGBS and limestone filler have a beneficial role on the chloride resistivity of concrete. The improvement in chloride resistance is related to a number of physical and chemical characteristics of these cements which result in changes in both microstructure and pore chemistry.

Concrete incorporating fine pozzolanic materials such as fly ash and GGBS offers an enhanced microstructure leading to enhanced permeation characteristics compared to conventional concrete. The higher densification of these materials acts as physical barriers to chloride ion ingress. However, the Portland cement used in this study had finer particles than those of pozzolanic cements. Besides, pozzolanic reaction occurs comparatively in slow pace. Therefore, the beneficial effect of fly ash and GGBS was evident in the later stages than earlier. In addition to their contribution to improved microstructure, the chloride binding capacity of these materials offers the chemical barrier to chloride ingress. Use of ultra-fine limestone filler showed a competitive performance at the earlier ages. However, the performance of Portland limestone concrete against chloride penetration did not change substantially with time.

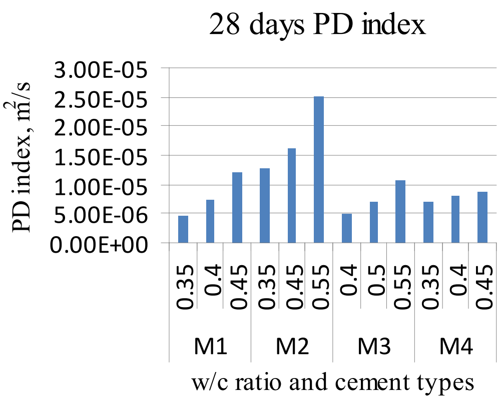

Figure 5: PD indices of 28 days old samples

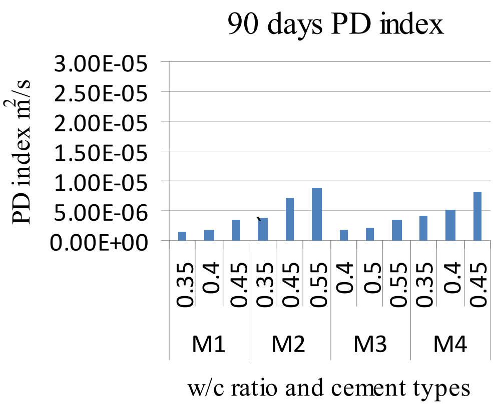

Figure 6: PD indices of 90 days old samples

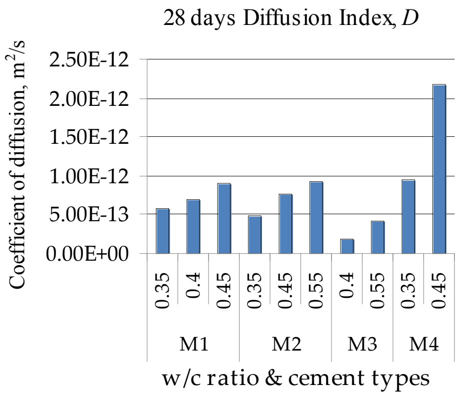

Figure 5: Diffusion indices of 28 days old samples

4.

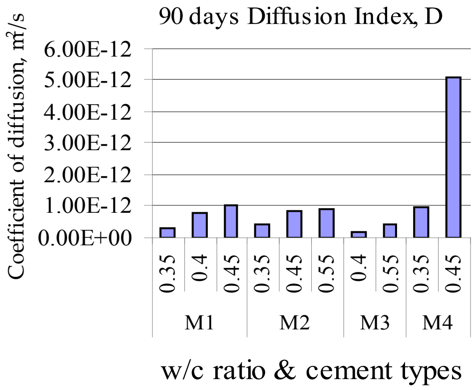

Figure 6: Diffusion indices of 90 days old samples

5. CONCLUSIONS

Electro-chemical migration of chloride ions through concrete is influenced by a number of factors. In order to examine the different influential factors, investigations on rapid chloride migration of concrete specimen with different cement types and w/c ratio was undertaken. Following conclusion can be drawn from the experimental results.

Two-cell tests are sensitive to w/c ratio i.e. chloride diffusivity changes with the change in w/c ratio of the concrete specimen. This effect was obvious for all cement types and at all ages. It was evident that higher chloride permeability of concrete was the consequence of increased w/c ratio which in turn was related to the microstructure of the concrete system.

An appropriate mix specification with the limitation on maximum w/c ratio can improve the aspect of durability against chloride penetration. A 2.5 folds decrease in PD index was recorded when the w/c ratio reduced from 0.45 to 0.35 for M1 concrete at 28 days.

In the early ages, M1 concrete performed well against chloride diffusion. Inclusion of limestone did not result in significant benefit. However, with time, fly ash and GGBS concrete showed higher resistance against chloride migration. The competitive behaviour of these pozzolanic cements was significant at later stages of hydration.

At equal strength grade, w/c ratio, cement content and age, GGBS concrete had the highest resistance against chloride among other cement types. The performance was more significant at the later stages.

PD test and MR tests are significantly different from methodological point of view. Conductivity based chloride measurement made MR test less laborious than PD test, which involved additional potentiometric titration to measure chloride concentration. However, the steady state condition was obtained faster in the PD test.

REFERENCES

Andrade, C. (1993). "Calculation of chloride diffusion coefficients in concrete from ionic migration measurements." Cement and Concrete Research, 23(3): 724-742.

Andrade, C., Sanjuán, M. A., Recuero, A. and Río, O. (1994). "Calculation of chloride diffusivity in concrete from migration experiments, in non steady-state conditions." Cement and Concrete Research, 24(7): 1214-1228.

British Standards Institution (2006). BS 8500-1:2006 Part 1: Method of specifying and guidance for the specifier. London, BSI.

Castellote, M., Andrade, C. and Alonso, C. (1999). "Chloride-binding isotherms in concrete submitted to non-steady-state migration experiments." Cement and Concrete Research, 29(11): 1799.

Castellote, M., Andrade, C. and Alonso, C. (2001). "Measurement of the steady and non-steady-state chloride diffusion coefficients in a migration test by means of monitoring the conductivity in the anolyte chamber. Comparison with natural diffusion tests." Cement and Concrete Research, 31(10): 1411-1420.

Dhir, R. K., El-Mohr, M. A. K. and Dyer, T. D. (1996). "Chloride binding in GGBS concrete." Cement and Concrete Research, 26(12): 1767.

Dhir, R. K., Jones, M. R. and Ahmed, H. E. H. (1990). "Determination of total and soluble chlorides in concrete." Cement and Concrete Research, 20(4): 579-590.

Dhir, R. K., Jones, M. R., Ahmed, H. E. H. and Seneviratne, A. M. G. (1990). "Rapid Estimation Of Chloride Diffusion-Coefficient In Concrete." Magazine Of Concrete Research, 42(152): 177-185.

Friedmann, H., Amiri, O., Aīt-Mokhtar, A. and Dumargue, P. (2004). "A direct method for determining chloride diffusion coefficient by using migration test." Cement and Concrete Research, 34(11): 1967.

Hardjito, D., Wallah, S. E., Sumajouw, D. M. J. and Rangan, B. V. (2004). "On the development of fly ash-based geopolymer concrete." ACI Materials Journal-American Concrete Institute, 101(6): 467-472.

Hornain, H., Marchand, J., Duhot, V. and Moranville-Regourd, M. (1995). "Diffusion of chloride ions in limestone filler blended cement pastes and mortars." Cement and Concrete Research, 25(8): 1667-1678.

Jain, J. A. and Neithalath, N. (2010). "Chloride transport in fly ash and glass powder modified concretes - Influence of test methods on microstructure." Cement and Concrete Composites, 32(2): 148-156.

Luo, R., Cai, Y., Wang, C. and Huang, X. (2003). "Study of chloride binding and diffusion in GGBS concrete." Cement and Concrete Research, 33(1): 1-7.

Mehta, P. K. (2001). "Reducing the environmental impact of concrete." Concrete international, 23(10): 61-66.

Papadakis, V. G. (2000). "Effect of supplementary cementing materials on concrete resistance against carbonation and chloride ingress." Cement and Concrete Research, 30(2): 291-299.

Sharfuddin Ahmed, M., Kayali, O. and Anderson, W. (2008). "Chloride penetration in binary and ternary blended cement concretes as measured by two different rapid methods." Cement and Concrete Composites, 30(7): 576-582.

Song, H. W., Lee, C. H. and Ann, K. Y. (2008). "Factors influencing chloride transport in concrete structures exposed to marine environments." Cement and Concrete Composites, 30(2): 113.

Stanish, K., Hooton, R. D. and Thomas, M. D. A. (2004). "A novel method for describing chloride ion transport due to an electrical gradient in concrete: Part 1. Theoretical description." Cement and Concrete Research, 34(1): 43-49.

Tang, L. and Nilsson, L.-O. (1992). "Rapid determination of the chloride diffusivity in concrete by applying an electrical field." ACI Materials Journal, 89(1): 40-53.

Truc, O., Ollivier, J. P. and Carcassès, M. (2000). "New way for determining the chloride diffusion coefficient in concrete from steady state migration test." Cement and Concrete Research, 30(2): 217-226.

Whiting, D. (1981). "Rapid measurement of the chloride permeability of concrete." Public Roads, 45(3): 101-112.

Yuan, Q., Shi, C., De Schutter, G., Audenaert, K. and Deng, D. (2009). "Chloride binding of cement-based materials subjected to external chloride environment - A review." Construction and Building Materials, 23(1): 1-13.

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BEHAVIOR OF FERROCEMENT AND RC COLUMN UNDER LATERAL AND AXIAL LOADING

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ABSTRACT

Structurally efficient building material should be lightweight, cost effective and especially the ones that can perform the desired functions. In the recent years a great deal of interest has been created in different sectors of engineering works on the potential application of Ferrocement. The purpose of this study is to investigate the behaviour of ferrocement and RC(Reinforced Concrete)column under lateral and axial loading. In this research work, three numbers of column specimens are constructed for both RC and ferrocement and properly cured for 28 days. Finally these specimens are tested accordingly. After observation of the test results it can be concluded that although RC column specimens indicate much higher percentage variation in case of total energy dissipation up to failure load but up to cracking load it shows comparatively lower percentage variation. Also in contrast of ductility, reserve strength, cracking load and failure load the RC column specimens indicates comparatively lower percentage variation with ferrocement column specimens, whereas in ferrocement column; 50% weighted equivalent reinforcement of RC column was used.

Keywords: Ferrocement, reinforced concrete, energy dissipation, ductility, cracking load, failure load.

1. INTRODUCTION

In multi-story reinforced concrete buildings the seismic inertia forces generated at its floor levels are transferred to the various structural elements to the ground Devastating earthquakes in the last decades have shown that non-engineered concrete frames are particularly vulnerable to seismic action and are the major causes of loss of lives. In earthquake resisting buildings design, particularly their main elements need to be built with ductility in them. Ductile buildings have the ability to sway back-and-forth during an earthquake and resistant earthquake effect with some damage but without collapse. For this purpose the study of building material that is structurally efficient but at the same time, should be lightweight, eco-friendly, cost effective and especially the ones that can perform the desired functions is very important. In that case ferrocement can be a potential solution in increasing the ductility, leading to increased slenderness and buckling resistance because in ferrocement construction the concrete and wire mesh are combined in such a manner that the advantages of both materials are utilized effectively. The subsequent concrete addition enables the buildings frame to limit the sway and lateral deflection.

I.Rosenthal (1986) and A. Winokur, and I. Rosenthal (1982) had demonstrated the possibility of using ferrocement as columns after conducting a preliminary test program. S. Razvi, and M. Saatcioglu (1989) investigated the behavior of small scale reinforced concrete columns specimens when Welded Wire Fabric (WWF) was used as lateral reinforcement to confine the concrete core of the column. Various combinations of WWF and tie reinforcement have been used as confinement steel. It was shown from the experimental results that, the use of WWF as confinement reinforcement improves concrete strength and ductility very significantly. Mo. Y. L and Wang S. J. (2000) conducted experiment on seismic behavior of RC columns with various tie configurations. M. Mansur and P. Paramasiva (1990) studied the behavior and strength of ferrocement box sections with and without concrete infill under axial and eccentric compression. It was concluded that ferrocement in the form of a box section in which reinforcing wire meshes are folded in the form of a cage may be used as a structural column, and its strength can be enhanced with concrete infill. Ferrocement encased short circular and square concrete columns with unreinforced and reinforced cores were investigated by S. K. Kaushik et al. (1990) it was observed that the ferrocement encasement increases the strength and ductility of columns for both axial and eccentric loading conditions. Another interesting research work was done by T.Ahmed et al. (1990), to investigate the possibility of using ferrocement as a retrofit material for masonry columns, the study demonstrated that the use of ferrocement coating strengthens brick columns significantly and improves their cracking resistance. A preliminary investigation into repair of short square columns using ferrocement was conducted by P.J. Nedwell et al. (1990), it was found that the use of ferrocement retrofit coating increased the apparent stiffness of the columns and significantly improves the ultimate load carry capacity. A proposed method presented by E. H. Fahmy et al. (1999), for repairing reinforced concrete columns using ferrocement laminates as a viable economic alternative to the highly expensive conventional jacketing methods, the experimental results demonstrated that irrespective of the pre-loading level or the mesh type, better behavior and load carrying capacity for all test specimens could be achieved compared to their original behavior. An investigation into the behavior and strength of reinforced concrete columns strengthened with ferrocement jackets was carried out by Abdullah and T. Katsuki (2003) and T. Katsuki and Abdullah (2001), the columns were tested under cyclic lateral forces and constant axial load. Test results showed that by providing external confinement over the entire length of the RC columns, the ductility is enhanced tremendously. Saatcioglu and Ozcebe (1989), tested full scale columns under slowly applied lateral load reversals. Test parameters were axial load, confinement reinforcement and deformation path. They found increased stiffness degradation and early strength degradation with addition of axial loads. The authors concluded that selection of a proper confinement. Elwood and Moehle (2003) observed that the lateral displacement or drift of a reinforced concrete column at axial failure was dependent upon and directly proportional to the spacing of transverse reinforcement and the axial stress developed within the column.

Almost all the structure whether industrial, commercial or housing are constructed of RCC. These structures are working nicely under normal circumstances, but in the event of major earthquake, higher load imposition etc. the structures may suffer permanent damage. In the recent years a great deal of interest has been created in different sectors of engineering works on the potential application of Ferrocement. In this consideration research on ferrocement is increasing day by day. In this present work the performance of conventional RC column is investigated in compared with Ferrocement column under lateral and axial loading.

2. METHODOLOGY

2.1 Test of the Materials

Structural elements exhibit their properties from their constituent materials such as aggregate, cement, reinforcement, water etc. These constituent materials work compositely yet retaining their own identity and qualitative characteristics. So, relevant test of the materials are important in order to know the properties of the structural elements.

2.1.1 Compressive Strength of Concrete

Three standard cylindrical specimens of concrete, 6 inch diameter and 12 inch height were casted in the moulds made of cast iron. After 24 hours the cylinders were remoulded and cured in water for 28 days. The mixing ratio of concrete was 1:1.5:3. Then the load was applied on the specimens by Universal Testing Machine (UTM) at convenient rate until failure according to ASTM C39.

Figure 1: Test Set up for Compressive Strength of Concrete

2.1.2 Yield Strength of the Reinforcement

In this study 10mm diameter reinforcements of BSRM (60 grade) were used to cast column specimen. Hence three rod specimens of 1 meter long for each diameter were cut and prepared for yield strength test. The test was carried out using Universal Testing Machine (UTM).

Figure 2: Test Set up for yield strength test of reinforcement

2.1.3 Fineness Modulus of the Sand

In the present work Sylhet sand was used as fine aggregate for the casting of the column specimens. The F.M. value was determined by using no. 100, no. 50, no. 30 no. 16, no. 8, and no. 4 sieves according to ASTM C33. For this purpose, the sand was dried to a constant temperature of 1100C for 24 hours in an oven and then 1200gm sample was taken for the determination of F.M. value.

2.2 Fabrication of the Testing Specimens

Six column specimens including three RC columns and three ferrocement columns were constructed for the test. For both types of column circular section were used and spiral were used as shear reinforcement.The specimens dimension were chosen 1000mm high and 152.4mm diameter in cross section. The bottom end of the column is fixed. The column base were designed and detailed to ensure the failure would occur above the column base. During the construction of specimens skilled labour and requisite quality control were achieved.

2.2.1 Specification of Reinforced Column Specimens

In reinforced column specimens 6 nos. of 10 mm diameter MS bar were used as longitudinal reinforcement which is equivalent to 3% gross concrete area and 3.3 mm diameter wire were used as shear reinforcement.

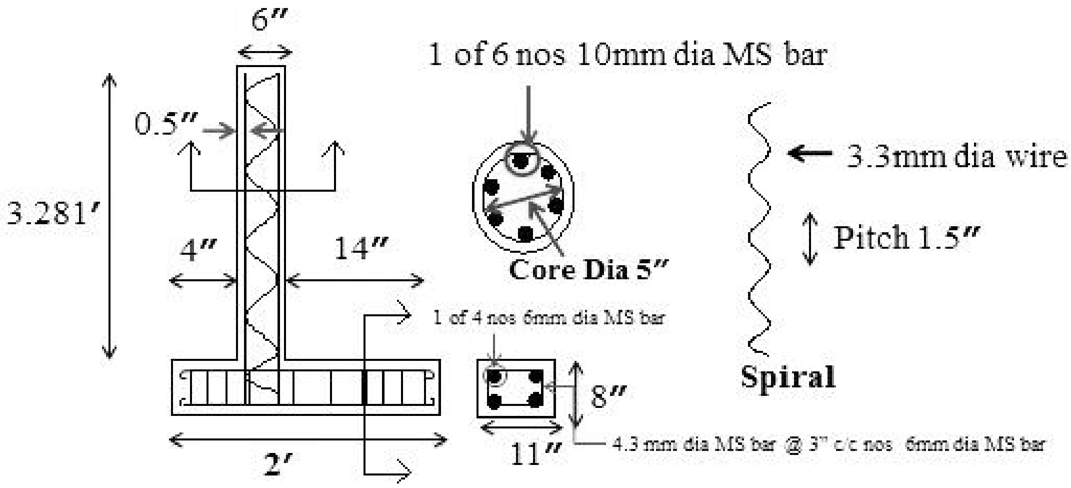

Figure 3: Typical reinforcement detailing of RC column

2.2.2 Specification of Ferrocement Column Specimens

In ferrocement column specimens, GI wire mesh having 12.7 mm opening were which is 50% weight equivalent of the reinforcement used in reinforced column. The dimensions were kept similar to the reinforced column.

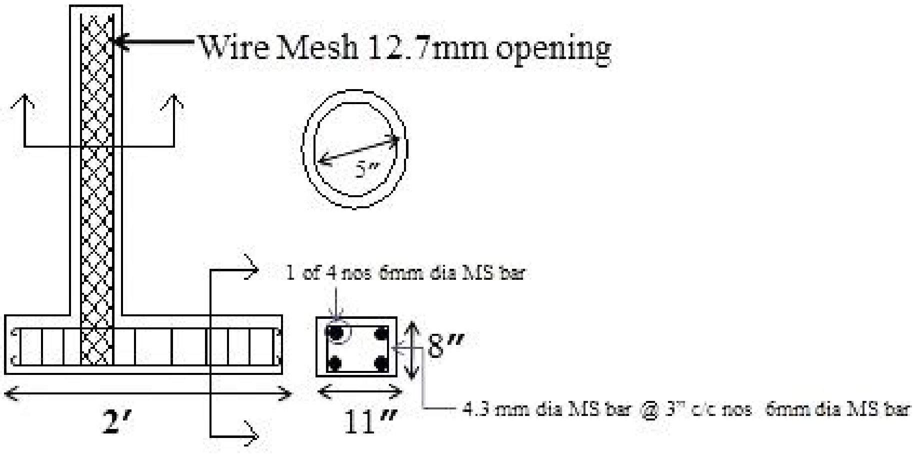

Figure 4: Typical reinforcement detailing of ferrocement column

2.2.3 Casting and Curing of the Column Specimens

Concrete and mortar mixing was done separately for the casting of RC and Ferrocement column respectively. For concrete mixing, Sylhet sand was used as fine aggregate which had F.M. value 2.96. As one third scale was taken, considering maximum size of coarse aggregate of 25mm crushed stone chips of 9.5mm downgraded were used as coarse aggregate. The mix ratio for concrete mixing was 1:1.5:3 by volume and the water cement ratio was taken as 0.56. For mortar mixing, Sylhet sand (F.M. = 2.96) and Ordinary Portland Cement was used and the cement-sand ratio was kept 1:2. In this present work, the column specimens were properly cured for 28 days using wet gunny bags.

Figure 5: Casting of the column specimens

2.3 Experimental Programme

The test was carried out on the column specimens using loading frame, hydraulic jack, DATA-LOGGER (TDS 303) and other accessories to determine the behavior of Ferrocement and RC column under lateral and axial loading.

2.3.1 Equipment Set up

The experimental setup consists of applying both horizontal and vertical load on the column specimens. A hydraulic jack supported on a horizontal steel beam was used to apply the incremental horizontal load and to apply constant axial load a hydraulic jack was set up on above the column which was supported on a steel beam (perpendicular to the axial loading direction) attached to the loading frame. However, there existed no means to subject the test specimens to a constant axial load concurrently with the incremental horizontal loading. Thus, after several revisions, an experimental setup was designed which included provision of several bearings in between the hydraulic jack (for applying axial load) and the steel beam that would allow the test specimens to undergo bidirectional loading which simulate the gravity and unidirectional seismic loading experienced by an actual column. To obtain precise information regarding load and deflection reading DATA-LOGGER (TDS 303) in addition with load cell and LVDT respectively were used during the test of the column specimens.

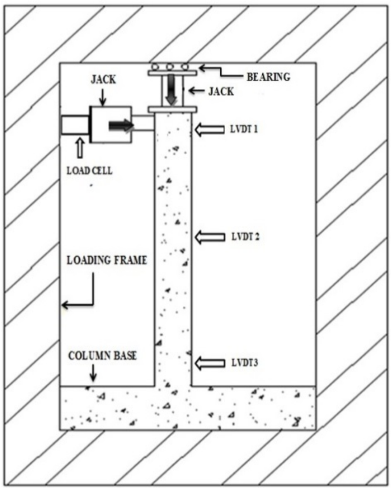

Figure 6: Schematic Diagram of Test Set up

2.3.2 Testing of the Specimens

The testing column specimen was at first mounted vertically inside the loading frame and the base was tightly fastened by using nuts and bolts as shown in Figure 7. A constant axial load of 5tonns was applied by means of vertical jack. For the simultaneous movement of the hydraulic jack and the column during the application of incremental horizontal load four bearing were provided in between the jack and the steel beam. A 50 ton capacity hydraulic jack along with a load cell mounted horizontally on a wooden frame between the loading frame and the column specimen. After that three LVDT were placed at the loading point, at the mid-point and at the bottom of the column specimen. Then the incremental horizontal load was applied and deflection measurements against the applied load at three LVDT points were simultaneously collected by means of DATA-LOGGER (TDS 303).

Figure 7: Laboratory Test Set up

3. RESULTS AND DISCUSSION

3.1 Material Properties

Table 1: Compressive Strength of Concrete

Serial No.	Diameter (inch)	Area (inch^2)	Observed Load, x (lb)	Actual load, y=0.9739x-1609.9 (lb)	Strength (psi)	Average strength (psi)	Failure Mode
1	5.90	27.34	100500	96268	3521	3507	C
2	5.95	27.81	103000	98702	3550		C
3	5.90	27.34	98500	94319	3450		C

NB: C=Combined Failure

Table 2: Yield Strength of Reinforcement

Nominal Diameter (mm)	SL No.	Weight (gm)	Length (cm)	Actual Diameter (mm)	Yield Load (kN)	Ultimate Load (kN)	Yield strength (MPa)	Ultimate strength (MPa)	Average Yield strength (MPa)
10	1	607.8	101.0	9.88	35	50.5	440	650	448
	2	611.1	101.2	9.90	37	51.0	465	654	
	3	611.5	102.0	9.86	35	50.0	440	646	

Table 3: Fineness Modulus of Sand

Sieve no. (ASTM)	Weight retained (gm)	Cumulative weight retained (gm)	Cumulative percent weight retained	Fineness Modulus(F.M)
No. 4	0.0	0.0	0.0	2.96
No. 8	89.7	89.7	7.5	
No. 16	326.2	415.9	34.7	
No. 30	369.4	785.3	65.4	
No. 50	278.8	1064.1	88.7	
No. 100	130.1	1194.2	99.5	
Total	1194.2		295.8	

3.2 Modes of Failure

The lateral load combined with the axial force lead to the cracking of concrete near the column base at first. Then with the progress of loading, further cracks developed at higher position of the column specimens.

Figure 8: Cracks in RC Column

Figure 9: Cracks in Ferrocement Column

3.3 Graphical Representation of the Test Results for RC and Ferrocement column

LVDT was placed at the top of the column specimens to measure the horizontal displacement corresponding to the horizontal load. Then horizontal load vs. deflection curve was plotted to obtain yield displacement, load corresponding to yield displacement, reserve strength, ductility and energy dissipation.

3.3.1 Deflection

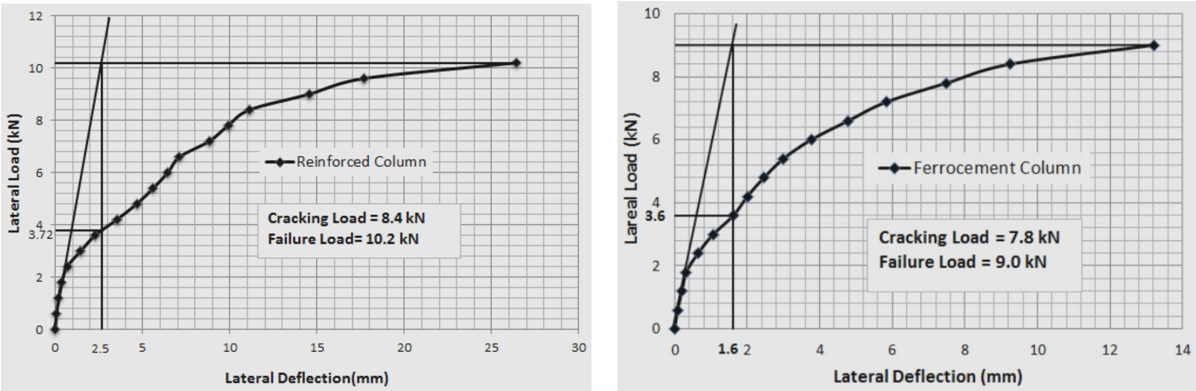

Figure 10: Lateral Load vs. Deflection Curve

In figure 10, typical load vs. deflection curve for reinforced concrete (RC 1) and ferrocement column (FC 1) is shown from which yield displacement and load corresponding to yield displacement were determined. In the following Table 4, the corresponding values for other specimens are shown.

Table 4: Cracking Load and Load Corresponding to Yield Displacement of Various Column Specimens.

Sample type	Sample designation	Cracking load (kN)	Avg. Cracking load (kN)	Failure load (kN) P_1	Avg. Failure load (kN)	Load corresponding to yield displacement (kN) P_2	Reserve Strength P_1/P_2	Avg. Reserve Strength (kN/kN)
RC	RC 1	8.4	8.2	10.2	10.2	3.72	2.74	2.80
	RC 2	7.8		9.80		3.60	2.72	
	RC 3	8.4		10.6		3.60	2.94	
FC	FC 1	7.8	7.8	9.00	9.0	3.60	2.50	2.48
	FC 2	8.4		9.20		3.70	2.49	
	FC 3	7.2		8.80		3.60	2.44	

3.3.1 Ductility of the Column Specimens

Ductility is the property of the structural element subjected to lateral load to sustain large inelastic deformation before failure. Ductility was quantified by the ratio of displacement at failure to the same displacement at the yield point.

Table 5: Ductility of the column specimens

Specimen type	Specimen designation	Ultimate displacement (mm)	Yield displacement (mm)	Ductility (mm/mm)	Average Ductility (mm/mm)
Reinforced Column	RC 1	26.42	2.5	10.57	10.34
	RC 2	23.44	2.2	10.65	
	RC 3	29.40	3.0	9.80	
Ferrocement Column	FC 1	13.20	1.6	8.25	8.12
	FC 2	14.60	1.8	8.11	
	FC 3	11.98	1.5	7.99	

3.3.2 Energy Dissipation up to Cracking Load

The amount of energy dissipated by the corresponding column specimen up to cracking load is the area covered in the Load-Deflection curve up to the crack load.

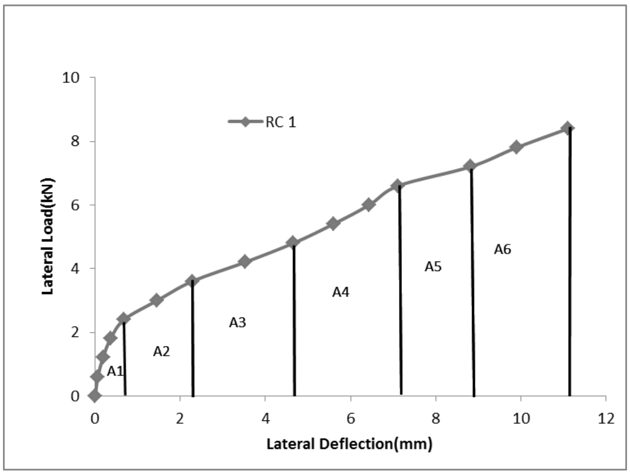

Figure 11: Energy Dissipation up to Cracking Load of Sample RC 1

Table 6: Energy Dissipation of the Column Specimens up to Cracking Load

Specimen type	Specimen designation	Energy Dissipation (kN-mm)	Average Energy Dissipation (kN-mm)
Reinforced Column	RC 1	59.30	57.31
	RC 2	42.56	
	RC 3	70.08	
Ferrocement Column	FC 1	40.27	43.54
	FC 2	58.24	
	FC 3	32.12	

3.3.3 Energy Dissipation up to Failure Load

The amount of energy dissipated by the corresponding column specimen up to failure load is the area covered in the Load-Deflection curve up to the failure load.

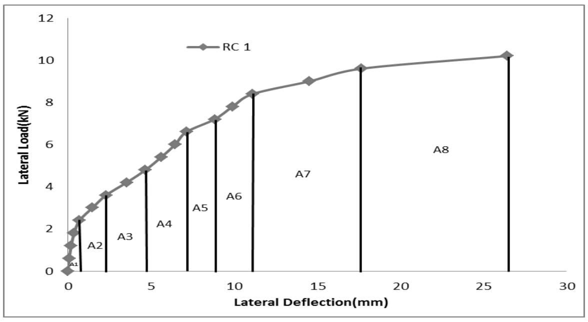

Figure 12: Energy Dissipation up to Failure Load of Sample RC 1

Similarly, the energy dissipations of the column specimens are calculated and tabulated in Table 7.

Table 7: Energy Dissipation of the Column Specimens up to Failure Load

Specimen type	Specimen designation	Energy Dissipation (kN-mm)	Average Energy Dissipation (kN-mm)
Reinforced Column	RC 1	205.66	201.11
	RC 2	176.92	
	RC 3	220.74	
Ferrocement Column	FC 1	88.81	93.56
	FC 2	116.42	
	FC 3	75.44	

4. CONCLUSIONS

Analysing the test results following conclusions can be drawn:

1. The average cracking load of RC column is 5% higher than the ferrocement column and the average failure load of RC column is 13% higher than the ferrocement column where the section property for both type of column is same as well as the loading condition.

2. The RC column shows larger lateral displacement than ferrocement column under similar loading condition and the average reserve strength of RC column is 13% higher than the ferrocement column.

3. In contrast of ductility, the RC column is 27% more ductile than Ferrocement column.

4. RC column dissipates 114% greater amount of energy than Ferrocement column up to failure load but up to crack load RC column dissipates only 31% higher energy than Ferrocement column.

After observation of the overall behaviour of ferrocement and reinforced column under lateral and axial loading it can be concluded that the RC column specimens indicate much higher percentage variation in case of total energy dissipation up to failure load but up to cracking load it shows comparatively lower percentage variation. Also in contrast of ductility, reserve strength, cracking load and failure load the RC column specimens indicates comparatively lower percentage variation with ferrocement column specimens, although in ferrocement column 50% weighted equivalent reinforcement of RC column was used. So, ferrocement column may be used as an alternative of RC column.

REFERENCES

A.Winokur, and I. Rosenthal (Oct. 1982), “Ferrocement in Centrally Loaded Compression Elements”, Journal of Ferrocement (Bangkok), V. 12, No. 4, pp. 357-364.

Abdullah and T. Katsuki((2003),“An Investigation Into the Behavior and Strength of Reinforced Concrete Columns Strengthened with Ferrocement Jackets”, Cement & Concrete Composites, V.25, pp 233-242.

ASTM C33 (Jan. 10, 1999), “Standard Practice for Fineness Modulus of Sand”, American Society for Testing and Material.

ASTM C39/C39M-05 (2005), “Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens”, American Society for Testing and Material.

E. H. Fahmy, Y.B.Shaheen and Y.S. Korany(April 1999), “Reparing Reinforced Concrete Columns using Ferrocement Laminates” Journal of Ferrocement, No. 2,Vol. 29.

Elwood, K.J. and Moehle, J.P(November 2003). “Shake Table Tests and Analytical Studies on the Gravity Load Collapse of Reinforced Concrete Frames”, PEER Report Series.

I. Rosenthal (July 1986), “Precast Ferrocement Columns”, Journal of Ferrocement (Bangkok), V. 16, No. 3, pp. 273-284.

M.Saatcioglu and G.Ozcebe (January 1, 1989), “Response of Reinforced Concrete Columns to Simulated Seismic Loading” ACI Structural Journal, V.86, pp. 3-12.

M. Mansur, and P. Paramasiva (September- October, 1990), "Ferrocement Short Column under Axial and Eccentric Compression", ACI Structural Journal, V. 87, No. 5, pp. 523-529.

Mo. Y. L and Wang S. J. (2000), "Seismic Behaviour of RC Columns with Various Tie Configurations", Journal of Structural Engineering, ASCE, Volume 126.

P.J. Nedwell, M.H. Ramesht and S. Rafei-Tanhanaki (1990), " Investigation into the Repair of Short Square Columns using Ferrocement", In Proceedings of the Fifth International Symposium on Ferrocement, Nedwell and Swamy (eds.), pp. 277-285.

S. Razvi, and M. Saatcioglu (September- October, 1989), "Confinement of Reinforced Concrete Columns with Welded Wire Fabric", ACI Structural Journal, V. 86, No. 5, pp. 615-623.

S. K. Kaushik, A. Prakash, and K.K. Singh (1990), "Inelastic Buckling of Ferrocement Encased Columns", In Proceedings of the Fifth International Symposium on Ferrocement, Nedwell and Swamy (eds.), pp. 327 – 341.

T. Ahmed, S. K. Ali and J. R. Choudhury (1990), "Experimental Study of Ferrocement as a Retrofit Material for Masonary Columns", In Proceedings of the Fifth International Symposium on Ferrocement, Nedwell and Swamy (eds.), pp. 269-276.

T. Katsuki and Abdullah (September-October, 2001), "Shear Strengthening of Reinforced Concrete Columns using Ferrocement Jacket", ACI Structural Journal.

LABORATORY INVESTIGATION OF NO FINES CONCRETE

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ABSTRACT

In many developed countries, the use of No finesor pervious concrete for the construction of pavements, car parks and driveways is becoming popular. In order to develop material specification for pervious concrete, it is necessary to conduct testing to evaluate the performance of this new type of high-performance concrete. The pervious concrete is produced by using cementitious materials, aggregates, and water. The effect of various types of aggregate and cement on the properties of pervious concrete was studied in this research. Performance of pervious concrete was evaluated in terms of air void, absorption, compressive strength, tensile strength and water permeability. Ninety six cylinders of 4″x8″ sizes were investigated. The porosity varies from 0.17 to 0.30.The water permeability varies from 5.5 mm/s to 13.2 mm/s. The tensile strength varies from 2 MPa to 4 MPa. The compressive strength varies from 5 MPa to 15 MPa. Tensile and compressive strength of pervious concrete was 40% less than that of the conventional concrete. Water permeability was found to be compromised for higher compressive strength.

Keywords: No fines or pervious concrete, compressive strength, porosity, permeability.

1. INTRODUCTION

Pervious concrete is a composite material consisting of coarse aggregate, Portland cement and water. It is different from conventional concrete as the mixture contains no fines in it. The aggregate is usually of a single size and is bonded together by a cement paste. The result is a concrete with a high percentage of interconnected voids that allow the penetration of water through the material matrix. Normal concrete has a void ratio around 3-5% and pervious concrete has higher void ratios from 18-40% depending on its application. Pervious concrete differs from normal concrete in several other ways. Pervious concrete has lower compressive strength, higher permeability and a lower density. Its compressive strength could be 65% lower than the normal concrete (Lee et al., 2009). Pervious concrete is increasingly being installed to improve stormwater quality and reduce runoff produced by urban settings. During the last few years, pervious concrete has attracted more and more attention in concrete industry due to the increased awareness of environmental protection.

Figure 7: Pervious concrete pavement (The Pacific Southwest Concrete Alliance, 2009)

Many laboratory and field studies have been conducted to investigate into various aspects of pervious concrete. Many studies revealed that unlike conventional concrete, the performance of pervious concrete is highly dependent on both concrete materials and construction techniques (Schaefer et al., 2006; Delatte et al., 2009; Delatte et al., 2007). The focus of pervious concrete technology is the balance of permeability and mechanical properties as well as durability. If the mixture is too wet and easy to compact, the voids will be clogged and the permeability will be compromised. If the mixture is too dry and hard for compaction, the pervious concrete pavement will be weak and vulnerable to various types of distress.

2. METHODOLOGY

- Literature review
- Materials selection (portland cement, water, uniform coarse aggregate).
- The basic mix proportion for no-fines pervious concrete is cement, coarse aggregate and water: 1.0:4.0:0.35 respectively. Ninety six cylinders of 4″x8″ sizes have been tested in the laboratory for different types of cement and aggregate.
- Determination of unit weight, air void, water permeability, compressive strength and tensile strength by laboratory performance.
- Explanation and comparison of various test results,

For defining the basic characteristics of pervious concrete, pervious mortar and pervious pavement systems, an experimental investigation was conducted to study the following properties: density, porosity, water permeability, compressive strength, and drying shrinkage. The materials, mixture proportions, measurements and test method used in this study are described in this section. Mix proportions for pervious concrete mixes, by weight as shown in table 1.

Table 6: mix proportions for pervious concrete mixes, by weight

Mix	OPC (%)	PCC (%)	W/c Ratio	CA/c Ratio
C1	100	0	0.35	4.0
C2	100	0	0.35	4.0
P1	0	100	0.35	4.0
P2	0	100	0.35	4.0

The basic mix proportion for no-fines pervious concrete is binder materials, coarse aggregate and water: 1.0:4.0:0.35 respectively. Mix c1 contained 100% ordinary portland cement and brick khoa. Mix c contained 100% ordinary portland cement and stone chips. Mix p1 contained 100% portland composite cement and brick khoa. Mix p2 contained 100% portland composite cement and stone chips.

2.1 Porosity

The porosity of the hardened concrete was calculated from the oven-dry and saturated weights, using the following equation (Park et al., 2004).

$$V_r = [1 - (\frac{W_2 - W_1}{\rho_w Vol}) x(100\%) \quad (1)$$

Where

ρ_w = density of water.
Vr = porosity
W_1 = weight under water
W_2 = oven dry weight
Vol = volume of sample,

2.2 Compressive strength

Compressive strength test was performed according to ASTM C39.For the pervious concrete, three cylindrical specimens 200 mm high and 100 mm in diameter were used. The specimens were capped with dental plaster on both loading surfaces. The specimens were cured in water (20°C) until the testing. The compressive strength reported is the average of three results taken from three identical cylinders. The laboratory test has shown in Figure 2 and Figure 3.

Figure 8: Compressive strength test of pervious concrete

Figure 9: Failure behaviour of compressive strength test of pervious concrete

2.3 Split-tensile strength test

The split-tensile strength test was conducted on three (3) cylinders from each mixture. A column plot was used to illustrate the comparisons and correlations of the split-tensile strengths to their uniformity coefficients in Figure 4 and Figure 5.

Figure 10: Split-Tensile Strength test of pervious concrete

Figure 11: Failure behaviour of split tensile strength test of pervious concrete

2.4 Water Permeability

For the pervious concrete, the constant head method was used to measure the water permeability. Figure 6 shows the schematic diagram of the permeability test. Two water heads were adopted, namely 700 and 715 mm.

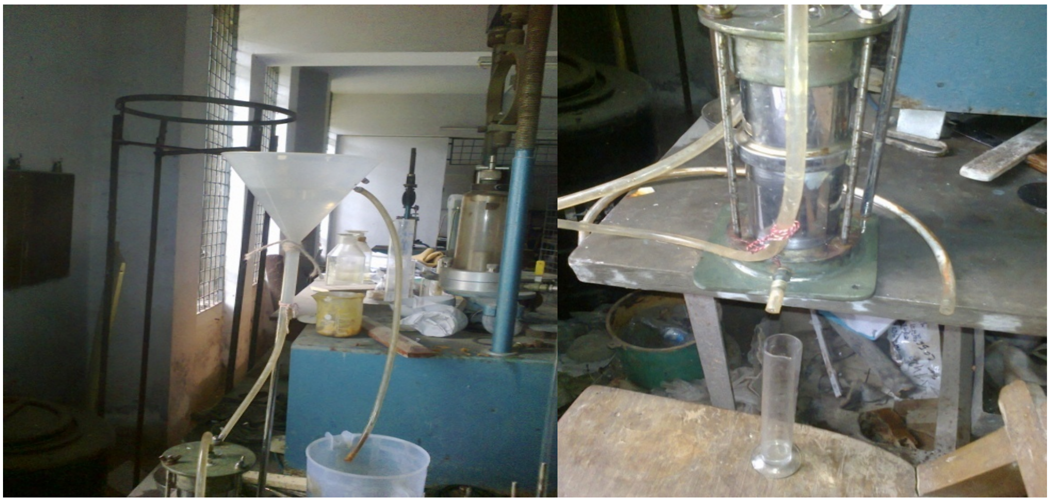

Figure 12: Constant head water permeability test of pervious concrete

3. RESULTS AND DISCUSSION

This section will extensively discuss the results of the experiments described in the previous section Comparisons will be provided of relevant relationships between water, aggregate and cement to show the influence each has on one another.

3.1 Compressive strength test

The specimens were tested at the age of 7 days, 14 days and 28 days for pervious concrete of ordinary portland cement and portland composite cement with brick khoa and stone chips.

The visible physical characteristics of the cylinders can provide preliminary information prior to subjecting the cylinders to any tests. For example, too much water in a mixture would cause the cement to sink to the bottom of the cylinder. The result would be clogging of the void spaces in the base of the concrete and reduce the permeability of water. Visually the bottom portion of the cylinder would be solid, there would be no voids. Higher compressive strengths and lower permeability rates can be expected from these cylinders due to the lack of void spaces. With the movement of the cement to the bottom of the cylinder, the top portion might be weaker than the bottom. Failure would begin at the top surface and work its way down the cylinder. The result might not be an abrupt failure but a long process in which the loading may actually increase after initially crushing the top and continue until the entire cylinder fails.

Cement that settled at the bottom of the cylinders in these mixtures is what gives the concrete its strength. Under real applications the water would have sent the cement completely through the aggregate and into the sub base, leaving the aggregate with little cement for bonding. Although a wide range of compressive strengths were obtained, none of the mixtures provide strength equal to that of conventional concrete. For the pervious concrete, the increases in rates of both mixes are quite similar. It can be noted that cement hydration has strong influence on conventional concrete properties.

The mean value of the compressive strength for 28 days of pervious concrete of brick khoa with 100% ordinary portland cement had 13 mpa, whereas pervious concrete of stone chips with 100% ordinary portland cement had 15 mpa. At the same age the mean value of the compressive strength of pervious concrete of brick khoa with 100% portland composite cement had 12.5 mpa, whereas pervious concrete of stone chips with 100% portland composite cement had 14.5 mpa. Figure 7 shows the development of compressive strength with age for ordinary portland cement with brick khoa (mix c1) and ordinary portland cement with stone chips (mix c2).mix c2 value is greater than mix c1. Figure 8 shows the development of compressive strength with age for portland composite cement with brick khoa (mix p1) and portland composite cement with stone chips (mix p2). As shown in figure 7 and figure 8, all concrete mixes showed improvement in strength with the increase in the age. The measured compressive strength varied considerably in all concrete mixes.

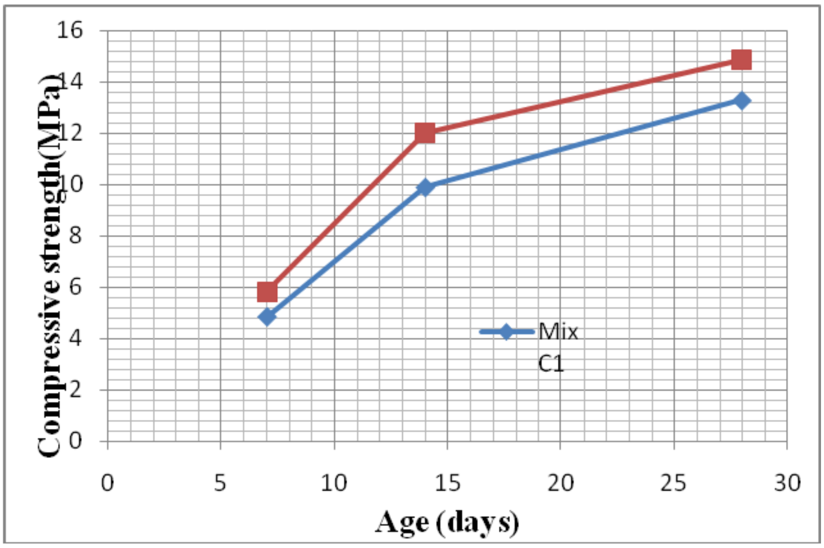

Figure 13: Development of compressive strength of Mix C1 and Mix C2

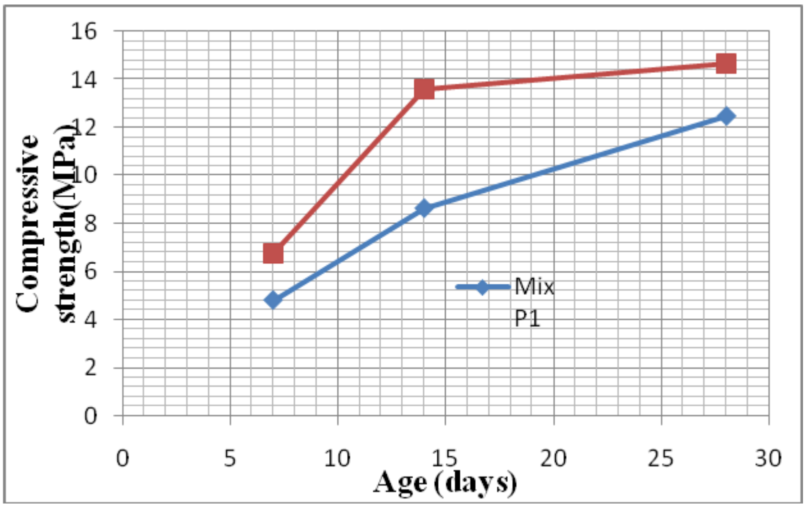

Figure 14: Development of compressive strength of Mix P1 and Mix P2

3.2 Splitting tensile strength

The splitting tensile strengths of concrete specimens were determined after 14 days of standard curing. The tests were carried out by splitting the cylinders in the machine used for compressive testing in accordance with bs 1881-117:1983. The results of splitting tensile strength tests are summarized in Table 2.

Table 7: Splitting tensile stress of pervious concrete at 14 days

	Ordinary Portland Cement (100%)		Portland composite cement (100%)	
	Brick Khoa	Stone Chips	Brick Khoa	Stone Chips
Mix	C1	C2	P1	P2
splitting tensile stress(MPa)	2	4	2	4
	2	4	2	4
	2	4	3	4
Mean(MPa)	2	4	2	4
Standard deviation	0	0	1	0

Table 2 summarizes the splitting tensile stress of pervious concrete at 14 days are represented of the Ordinary Portland cement and Portland composite cement between brick khoa and stone chips. In table 2, the mean value of the tensile strength for 14 days of pervious concrete of Brick khoa with 100% Ordinary Portland cement (mix C1) had 2 MPa, whereas pervious concrete of stone chips with 100% Ordinary Portland cement (mix C2) had 4

MPa. At the same age the mean value of the tensile strength of pervious concrete of Brick khoa with 100% Portland Composite cement (mix P1) had 2 MPa, whereas pervious concrete of stone chips with 100% Portland Composite cement (mix P2) had 4 Mpa. There is a smaller reduction in splitting tensile strength compared to the reduction in the compressive strength.

3.3 Water permeability

Permeability rates obtained in this experiment are also consistent with what prior researchers have found. Although a wide range of permeability rates were seen from this experiment, they are not typically the limiting factor. Water flow through pervious concrete is usually restricted by the permeability rates of the soil beneath the concrete. Higher permeability rates in pervious concrete is advantageous as it allows for clogging of the void spaces without being detrimental to the flow of water through the concrete.

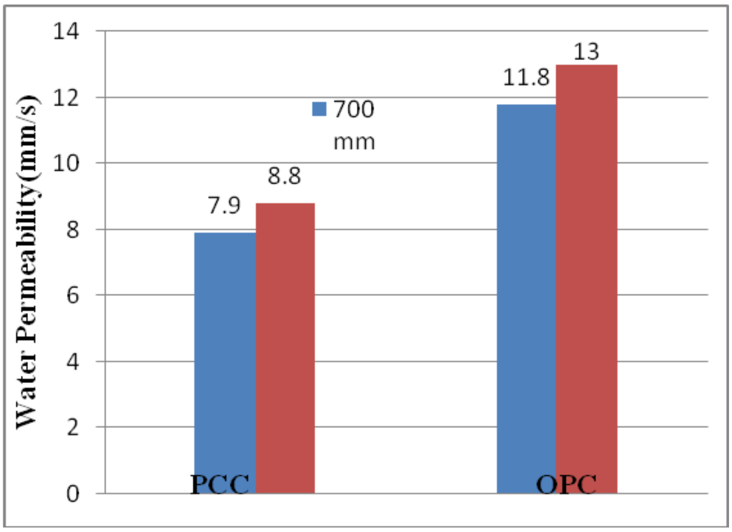

Figure 15: Effect of water head and on the permeability of pervious concrete (Mix C1 & P1)

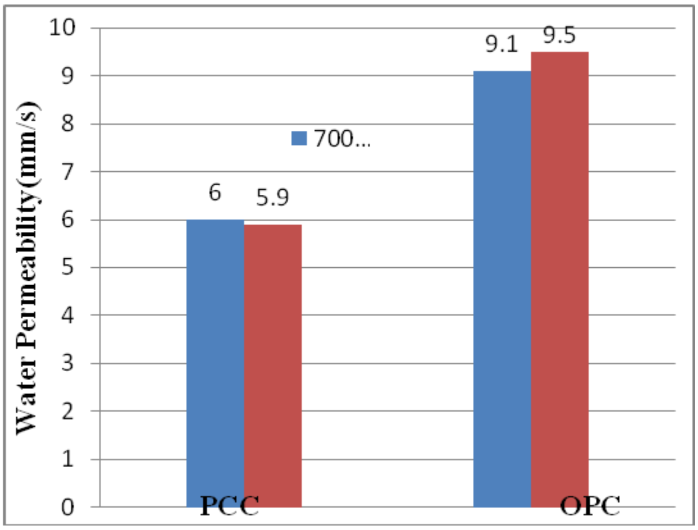

Figure 16: Effect of water head and on the permeability pervious concrete (Mix C2 & P2)

The water permeability under 700 mm water head by 100% PCC and OPC of pervious concrete is between 7.5 mm/s to 13.2 mm/s for brick khoa. The water permeability under 700 mm water head for 100% PCC and OPC of pervious concrete is between 5.5 mm/s to 10.9 mm/s for stone chips. The water permeability of pervious concrete is significantly influenced by pore structure which is affected by compaction and grading.

Permeability rates in the first mixtures are considerably less than the later mixtures. Permeability rates are also relatively consistent with compaction and density. Higher compaction energies increase the density thereby reducing the porosity of the concrete. The reduction in porosity leads directly to a reduction in the permeability rate. The mean water permeability under different water heads for each pervious concrete mixes is shown in Figure 9 and Figure 10.

The water permeability of pervious concrete is relatively high, especially for pervious concretes by 100% Ordinary Portland cement and Portland Composite cement. Sometimes the water permeability coefficient under 715 mm water head showed the lowest value for pervious concrete mixes, while the water permeability under 700 mm water head had the highest value for pervious concrete with 100% Ordinary Portland cement and Portland Composite cement. This is an unexpected result, since higher water permeability is expected under the highest water head of 715 mm. This is probably caused by change in water flow pattern from steady to turbulent. The sizes of pores in pervious concrete are large; hence high water head may lead to turbulent flow.

3.4 Relationship between porosity and water permeability

Figure 11 shows the relationship between porosity and water permeability for pervious concrete mixes. The water permeability decreased significantly when the porosity is decreased. The water permeability increased significantly when the porosity is increased. The water permeability is around 7.78 mm/s for porosity 0.29. The range of water permeability of pervious concrete is large for all mixes under two water heads due to the sensitivity of water permeability of pervious concrete to pore size and pore structure.

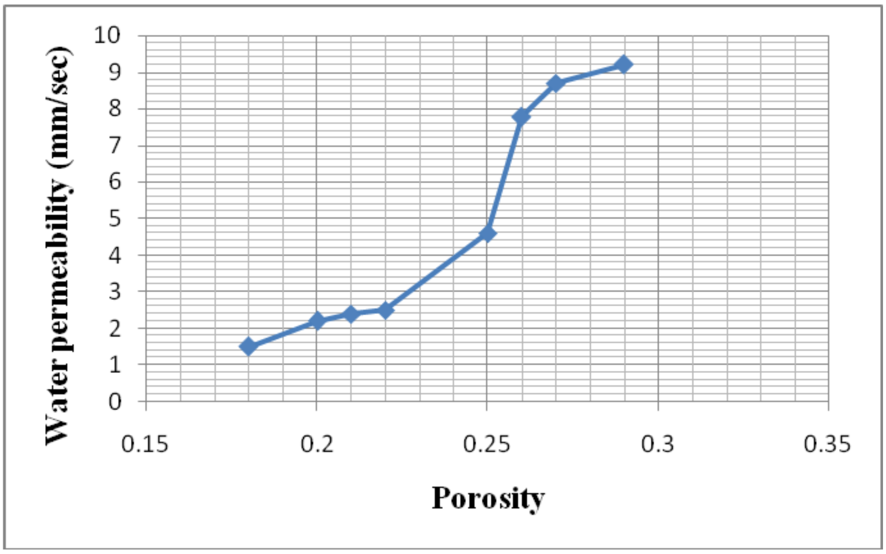

Figure 17: Relationship between porosity and water permeability for pervious concrete

4. CONCLUSIONS

No fines concrete has high water permeability due the presence of interconnected air voids. The presence of high porosity relative to conventional concrete makes the pervious concrete to become light weight concrete with limited compressive strength. However, pervious concrete has been significantly popular for a few decades due to its potential to reduce the incidence of flooding, and to assist in recharging the ground water level.

The porosity of the pervious concrete was 0.24, compared to 0.08 for conventional concrete. The porosity of pervious concrete was not significantly influenced by age. The compressive strength of the pervious concrete was around 11 MPa. The weight loss for pervious concrete on air drying was twice larger than that for conventional concrete. No fines concrete, although not as strong as conventional concrete, provides an acceptable alternative when used in low volume and low impact areas. Strength is sacrificed for permeability but not to any degree which would render the pervious concrete non-functional.

REFERENCES

Lee Yee Loon, Goh Kau Soon, Koh Heng Boon, Ismail Bakar, Foamed Aggregate Pervious Concrete – An Option for Road on Peat. Proceedings of MUCEET2009 Malaysian Technical Universities Conference on Engineering and Technology, June 20-22, 2009: p.1,26-29.

Delatte N, Mrkajic A, Miller DI. Field and laboratory evaluation of pervious concrete pavements. Transport Res Rec 2009;2113:132–9.

Delatte, N., Miller D, Mrkajic A. Field performance investigation on parking lot and roadway pavements (final report). Silver Spring (MD): RMC Research & Education Foundation; 2007.

Schaefer, V.R., Wang K, Suleiman MT, Kevern JT. Mix design development for pervious concrete in cold weather climates. Final report, Ames (IA): National Concrete Pavement Technology Center, Iowa State University; 2006.

The Pacific Southwest Concrete Alliance, 2009, Pervious concrete viewed 10 January 2009, <http://www.concreteresources.net/>

NECESSITY OF NON-LINEAR BOND STRESS-SLIP MODEL FOR SIMULATING THE CYCLIC BEHAVIOR OF REINFORCED CONCRETE (RC) MEMBERS

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ABSTRACT

A possible source of seismic failure in existing reinforced concrete (RC) structures is loss of anchorage in column reinforcement, along deficient lap splices with short lap length and inadequate transverse reinforcement conditions. Perfect bond assumption between steel and concrete in this case will not simulate the behaviour of RC members accurately under reverse cyclic loading. Reliable modelling of the bond slip behaviour and anchorage failures in such columns is important for performance assessment of existing buildings using nonlinear static and dynamic analysis methods. Numerous experimental and analytical examples are presented in this study from where it will be clearly understood that without incorporating the cyclic bond stress slip model in RC members, the complete analytical RC member model does not able to capture important response characteristics associated with the cyclic behaviour of reinforced concrete member with lap splices.

Keywords: Bond-stress, slip, reinforced concrete, modellin.

1. INTRODUCTION

One source of severe seismic damage in poorly-detailed reinforced concrete buildings is the loss of anchorage between reinforcing bars and concrete along short and poorly-confined lap splices in columns, which are located typically above floor levels where large inelastic demands are expected. Typical lap splice lengths of 20 to 30 longitudinal bar diameters, which are commonly encountered in many existing and poorly-detailed reinforced concrete buildings worldwide, have been shown to be inadequate in transferring the tensile stresses in longitudinal reinforcement along the lap splice region of a column. The load-deformation responses of columns representative of those found in older buildings are not well understood; and in particular, the degradation of strength and stiffness of a column due to splice failure and the ability of the column to undergo inelastic deformation while maintaining axial load capacity, are of interest. The effects of bond deterioration and slip deformations in longitudinal reinforcing bars on the overall response of reinforced concrete columns with inadequate lap-splice lengths must be taken into account in order to develop reliable analytical modeling approaches for such lap-splice-deficient columns, particularly for improvement of nonlinear analysis methods used for seismic performance assessment of existing buildings.

A significant number of experimental and analytical studies have been conducted on the anchorage and bond slip characteristics isolated bars embedded in concrete. Several constitutive bond stress vs. slip relationships have been proposed to simulate both pullout and splitting modes of bond failures for individual bars. The current state-of-the-art modeling approach for simulating bond slip behavior in reinforced concrete columns with deficient lap splices consists only of incorporating simple zero-length moment vs. slip rotation springs at the splice regions of column members, which are intended to represent deformations associated with bond slip at critical locations where inadequate anchorage conditions are provided. In general, only monotonic bar pullout vs. slip and moment vs. curvature analyses are employed, in order to generate a moment vs. rotation envelope for the bond slip spring, and the cyclic behavior of the bond slip spring is represented via predefined and somewhat ad-hoc unloading and reloading rules (e.g., Cho and Pincheira, 2006). A number of micro and macro analytical modeling approaches for simulating the bond slip behavior in reinforced concrete structural members are available in the literature. Out of all modeling approaches, macro fiber modeling approach by Chowdhury and Orakcal (2012) which involves modifying the formulation of the MVLEM (Multi Vertical Line Element Model) is found effective in simulating the bond slip behavior of reinforced concrete member. Numerous experimental and analytical examples are presented in this study from where it will be clearly understood that without incorporating the cyclic bond stress slip model in RC members, the complete analytical RC member model does not able to capture important response characteristics associated with the cyclic behavior of reinforced concrete member with lap splices, the response of which is governed by either bond slip, flexure, or a coupled combination thereof.

2. RESPONSES OF RC MEMBERS WITH INADEQUATE SPLICE GOVERNED BY BOND-SLIP FAILURE

Chowdhury and Orakcal (2012), and Chowdhury (2011)] simulated few column specimens tested by Melek and Wallace (2004) where most of the column specimens failed due to bond (splice failure). In Chowdhury (2011) model formulation, the coupled axial-flexural response of concrete was represented using a series of uniaxial concrete elements that were connected to rigid beams at the top and bottom of each model element, enforcing the plane-sections-remain-plane kinematic assumption on concrete. Uniaxial steel elements were connected to the rigid beams, and therefore to concrete, through uniaxial bond slip springs located at top level of each model element. The advanced constitutive relationship proposed by Chang and Mander (1994) was implemented in the analytical model for concrete, since it allows details calibration of the monotonic and hysteretic parameters, for improved representation of concrete stress vs. strain behavior. The constitutive model adopted for reinforcing steel used was the well-known Menegotto and Pinto (1973) relationship, as extended by Filippou et al. (1983) to include isotropic strain hardening effects. For representing the bond stress vs. slip deformation behavior of the bond slip springs, two constitutive models were implemented. In order to address the possibility that the transverse reinforcement would restrain the widening of splitting cracks close to the tie locations, bond slip springs in the vicinity of ties were assigned a hysteretic constitutive bond stress vs. slip relationship representing pull-out type bond slip behavior in confined concrete (Eligehausen et al. (1983), Fig. 1(b)), whereas the springs between ties were assigned a hysteretic constitutive relationship representing splitting-type bond slip behavior in partially-confined concrete (Harajli (2009), Fig. 1(a)). Details of the analytical model formulation are provided in Chowdhury and Orakcal (2012), and Chowdhury (2011).

Fig. 3 compares the measured and predicted lateral load vs. top displacement responses for four of the six column specimens. It can be observed in Fig. 3 that all of the specimens exhibit similar responses, with sudden degradation in lateral load, initiated by bond slip failure along the lap splice, at drift levels ranging between 1% and 1.5%. Overall, the analytical model results are in close agreement with the experimental measurements, for all six column specimens tested, regardless of the axial load level, column height, or loading history (Chowdhury (2011)). The model provides a reasonably accurate prediction of the overall lateral load – displacement response characteristics of the column specimens; including the initial stiffness, the lateral load capacity, post-peak degradation behavior in lateral load and cyclic stiffness, and pinching properties of the response.

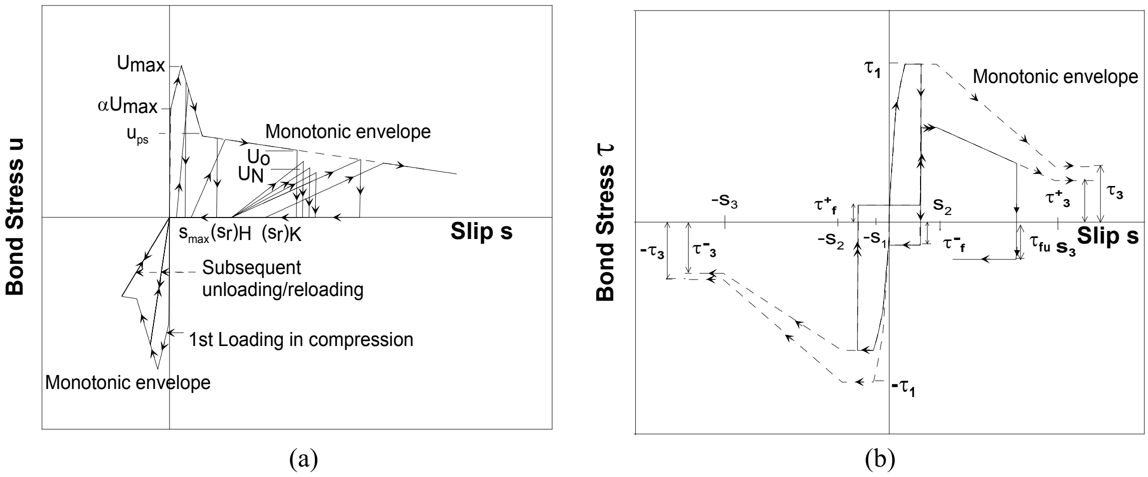


Figure 1: Hysteretic Bond Stress vs. Slip Constitutive Relationships for (a) Splitting (b) Pullout

The column specimens tested by Melek and Wallace (2004) were used to experimentally calibrate and evaluate the accuracy of the analytical model proposed by Chowdhury and Orakcal (2012). In this experimental program, a total of six full-scale column specimens were tested. Specimen heights ranged between 1.52 and 1.83 meters (5 and 6 feet), each with a full-scale 457x457 mm (18x18 in.) square cross section. The reinforcement configurations of the specimens shown in Fig. 2, with eight 25.4 mm (1 in.) nominal diameter vertical bars, and 9.5 mm (0.37 in.) diameter hoops with 90-degree hooks spaced at 305 mm (12 in.) on center, were based on a review of typical reinforcing details in older buildings of pre-1970 construction in the United States. A lap splice length of $20d_b$ (20 longitudinal bar diameters) was used in the construction of specimens. During testing, axial load was held constant, and a prescribed cyclic lateral displacement history was applied at the top of each specimen. The objective of three of the tests conducted on specimens 2S10M, 2S20M, and 2S30M was to assess the influence of axial load on columns with short lap splice lengths and widely-spaced transverse reinforcement, under moderate average shear stress levels. A detailed description of the experimental program can be found in Melek and Wallace (2004). Only illustrative comparisons between model predictions and experimental measurements are presented in Fig. 3, whereas all of the response comparisons are available in Chowdhury (2011).

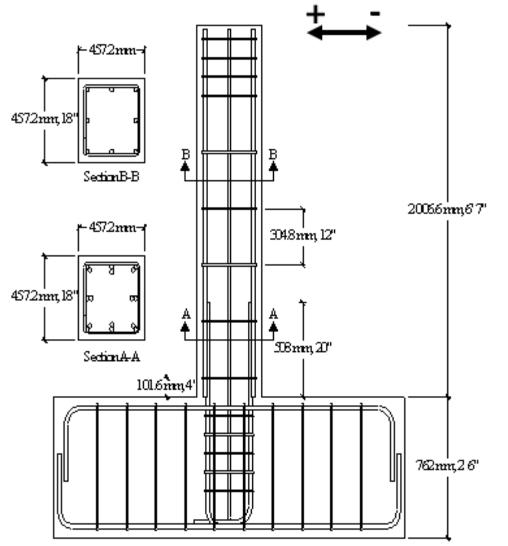


Figure 2: Column Specimen Details (Melek and Wallace [4])

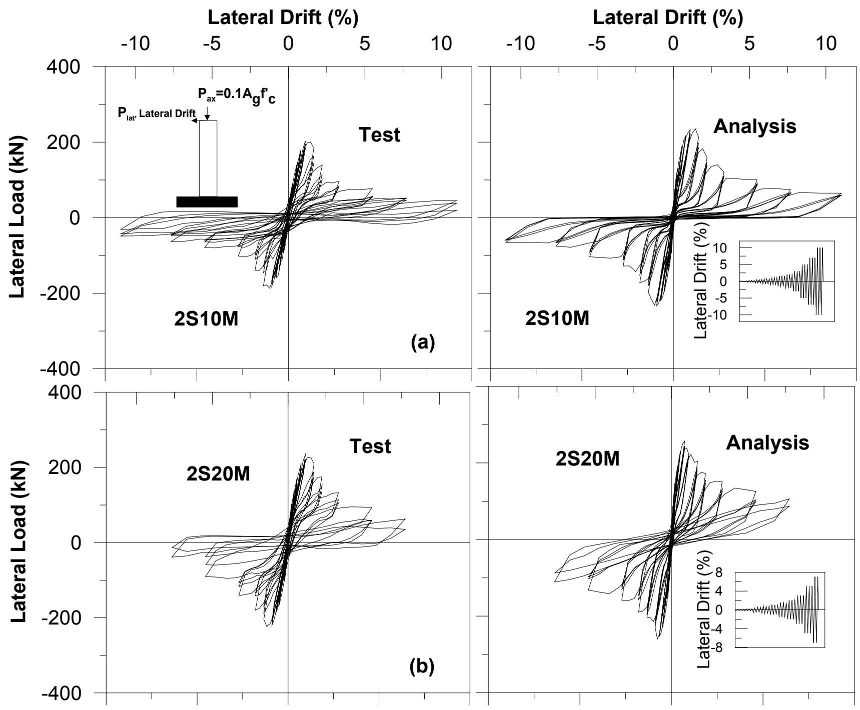


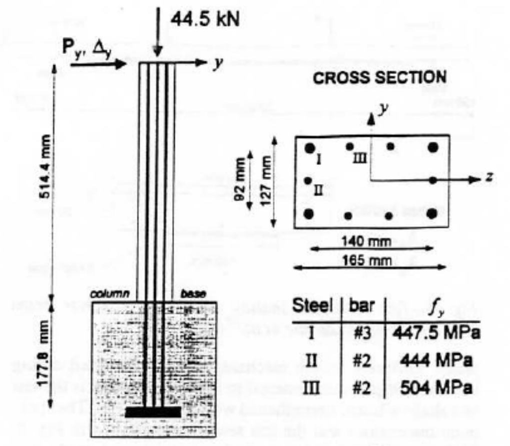
Figure 3: Measured and Calculated Lateral Load vs Displacement Responses for Column Specimens (a) 2S10M, (b) 2S20M (Chowdhury [3])

3. RESPONSES OF RC MEMBERS WITH CONTINUOUS REINFORCEMENT GOVERNED BY BOND-SLIP DEFORMATION

In this section two column specimens with continuous longitudinal reinforcement tested by Low and Moehle (1995) and Bousias et al. (1995) along-with analytical simulation made by Chowdhury (2011) is discussed to illustrate the strain penetration effect in the anchorage zone.

One of the columns tested by Low and Moehle (1987) , labeled Low-Moehle “Specimen 1”, with continuous longitudinal reinforcement, was selected by Chowdhury (2011) for investigating the efficiency of the model in simulating strain penetration effects. The specimen geometry is shown in Fig. 4. The anchorage length of the longitudinal reinforcement in the specimen foundation (pedestal) was 178mm, corresponding to 23 longitudinal bar diameters. Fig. 6 presents three different analytical results obtained using Chowdhury (2011) model. The solid line represents results of the first modeling approach (Model 1), where bond slip deformations in both the column and in the anchorage zone (strain penetration) are considered. The second model (Model 2), the results of which are represented by the dashed line with narrow spacing, considers bond slip deformation in the column only, neglecting the strain penetration effects. In the third model (Model 3), represented by the dashed line with wide spacing, all bond slip and strain penetration effects are neglected (perfect bond condition). All three models capture the column lateral load capacity accurately. In Models 2 and 3, the pre-yield stiffness in the analytical results is overestimated, since column base rotations due to the strain penetration effects are ignored. Moreover, the models that do not consider column bond slip and strain penetration (model 2 and model 3) obviously tend to overestimate the hysteretic energy dissipation (cumulative area under the load – displacement loops) of the specimen. From Fig. 6, it can be deduced that the hysteretic behavior of reinforced concrete columns, subjected to severe seismic excitations, is dependent on the bond interaction between steel and concrete, even if no anchorage failure takes place.

Fig. 5 illustrates details of the column specimen tested by Bousias et al. (1995). The specimen was 1490 mm long, with a cross section of 250 mm by 250 mm, and anchorage length of 480 mm (30db) in the foundation. Similarly with the ‘Specimen 1’by Low and Moehle (1987), three models were generated using the present analytical model by Chowdhury (2011) for this specimen, the predictions of which are presented in Fig. 7. The model formulation considering bond slip deformations in the both column and in the anchorage zone accurately predicts the cyclic attributes of the measured response, including lateral load capacity, shape of the hysteretic loops, cyclic stiffness degradation, cyclic energy dissipation capacity, and pinching behavior.



Steel	bar	f_y
I	#3	447.5 MPa
II	#2	444 MPa
III	#2	504 MPa

Figure 4: Geometry and Loading Conditions for Specimen 1’ (Low and Moehle [10]).

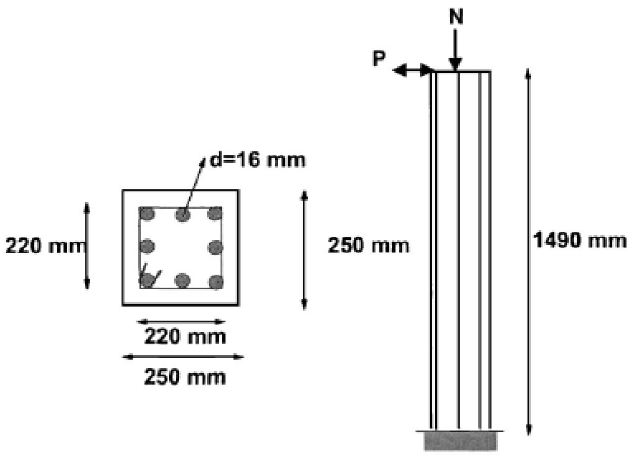


Figure 5: Bousias Specimen (Bousias et al. [11]).

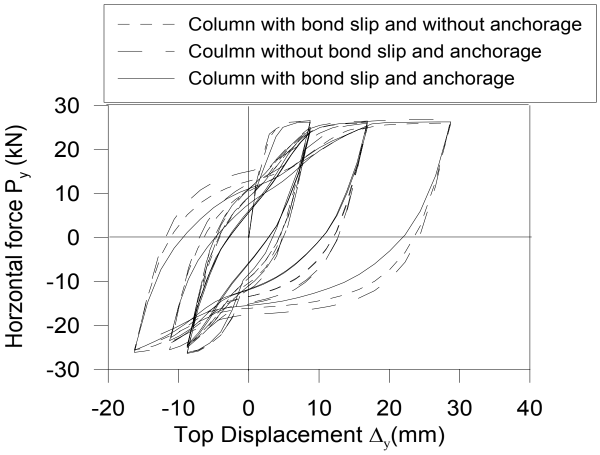


Figure 6: Analytical Response Prediction for 'Specimen 1′.

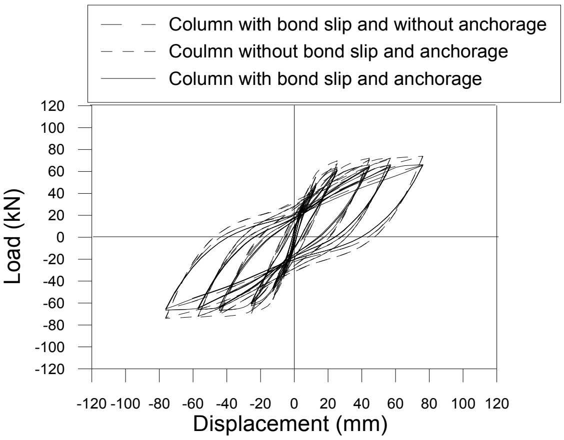


Figure 7: Analytical Response Prediction for 'Bousias et al. [11]' Specimen

4. CONCLUSIONS

Without non-linear hysteretic bond stress slip model it is not possible to provide reasonably accurate representations of important global response of reinforced concrete columns with inadequate lap splices; including lateral load capacity, rapid degradation in lateral load due to splice failure, ductility, cyclic stiffness characteristics, and pinching properties. Realistic consideration of bond slip deformations and anchorage failures is very much necessary to get proper seismic response and performance of reinforced concrete structures. The RC model with bond stress slip model is able to show accurately the influence of bond slip deformations in the anchorage zone (strain penetration effects) on the global cyclic response and pinching characteristics of columns with continuous longitudinal reinforcement. Implementation of hysteretic bond-stress slip model along with steel and concrete model into a computational platform will provide design engineers improved analytical capabilities to represent the seismic behavior of splice-deficient columns and anchorage failures, which is essential for the application of performance-based evaluation methods for existing structures.

REFERENCES

Bousias, S.N., Verzeletti, G., Farids, M.N. and Guitierrez, E..(1995), "Load-Path Effects in Column Biaxial Bending with Axial Force. Journal for Enviromental Engineerin, 125(5):596-605.

Cho, J.Y. and Pincheira, J.A. (2006), "Inelastic Analysis of Reinforced Concrete Columns with Short Lap Splices Subjected to Reversed Cyclic Loads". ACI Structural Journal,, 103(2): 280-290.

Chowdhury, S. R. and Orakcal, K. (2012), "An Analytical Model for Reinforced Concrete Columns with Lap Splices", Engineering Structures , V. 43: 180-193.

Chowdhury, S.R.(2011), "Analytical Modeling of Reinforced Concrete Columns with Lap Splices"' PhD Dissertation (2011), Department of Civil Engineering, Bogazici University, Istanbul, Turkey

Chang, G. A. and Mander, J. B.(1994), " Seismic Energy Based Fatigue Damage Analysis of Bridge Columns: Part I—Evaluation of Seismic Capacity", *NCEER Technical Report* No. NCEER-94-0006, State University of New York, Buffalo, NY, 222 pp.

Eligehausen, R., Popov, E.P. and Bertero, V.V.(1983), "Local Bond Stress-Slip Relationships of Deformed Bars Under Generalized Excitations", Report No. UCB/EERC 82-23, University of California, Berkeley, CA., U.S.A.

Filippou, F.C., Popov, E.G. and Bertero, V.V.(1983), " Effects of bond deterioration on hysteretic behavior of reinforced concrete joints", EERC report no. UCB/EERC-83/19, Earthquake Engineering Research Center, University of California, Berkeley, CA, USA.

Harajli, M.H.(2009), " Bond Stress-Slip Model for Steel Bars in Unconfined or Steel, FRC, or FRP Confined Concrete under Cyclic Loading", Journal of Structural Engineering, ASCE, 135(5): 509-518.

Low, S.S. and Moehle. J.P. (1987) "Experimental Study of Reinforced Concrete Columns Subjected to Multi-Axial Cyclic Loading", EERC Report 87/14. Earthquake Engineering Research Center, University of Calirfornia, Berkeley.

Melek, M. and Wallace, J. W.(2004), "Cyclic Behavior of Columns with Short Lap Splices", ACI Structural Journal , 101(6): 802-811.

Menegotto, M and Pinto, E. (1973), "Method of analysis for cyclically loaded reinforced concrete plane frames including changes in geometry and non- elastic behavior of elements under combined normal force and bending", IABSE symposium on resistance and ultimate deformability of structures acted on by well-defined repeated loads, Lisbon, Portugal.

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STRENGTH DETERIORATION OF BRICK AGGREGATE CONCRETE IN NACL ENVIRONMENT

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ABSTRACT

The use of concrete as a constructional material has been well established all over the world for its versatile uses. Normally concrete made of stone chips or brick chips attains its strength with time unless it is attacked by aggressive environmental spices. Sea water has a total salinity of 3.5% of which 78% of the dissolved solid of sodium chloride. Concrete often comes in contact with salt ions when exposed to sea water / marine environments and suffers the deterioration due to the reaction of salt ions with the hardened cement matrix. The paper covers an experimental study on the short term effects of sodium chloride environments on compressive and tensile strength development of brick aggregate concrete. A total of 216 nos. specimens (6"×12" cylinder) were cast from a particular concrete mix (1:2:4) using different w/c ratios (0.45, 0.50, 0.55) and were cured in NaCl solution of different concentration (3%, 5% & 10%). The test specimens were also cured in plane water (PW) to use as control specimens to evaluate the deterioration of the salt affected concrete. Both compressive & tensile strength of the concrete specimens were evaluated at the end of different exposure periods (7, 14 & 28 days). Enhanced salt concatenations of the curing solution were used as an accelerated test technique so as to get the effects of salt ions on concrete strength within short time. Some important information regarding the strength deterioration of concrete in sodium chloride environment was obtained. The rate of strength deterioration was found to increase with the increase of salt concentration and water cement ratio for a particular curing period; the maximum being 34% for w/c = 0.55. Loss of tensile strength was reported to be more as compared to compressive strength which may be due to the formation of micro cracks and their subsequent propagation with time.

Keywords: Sodium Chloride, Compressive Strength, Tensile Strength, Brick Aggregate Concrete, Strength Deterioration.

1. INTRODUCTION

Concrete has been the most widely used constructional material all over the world since the long time. The quality of concrete is dependent on several factors such as gradation and moisture content of aggregate, water/cement ratio, proportioning and method of mixing, placing, compaction, curing etc. Again the concrete life may be reduced by the deteriorating effects of two major causes; firstly due to weathering effect such as alternate wetting and drying, alternate freezing and thawing, chemical action, temperature variation, aggressive salt action etc.; secondly by mechanical action such as cyclic loading, wear, abrasion etc.

From the early stages of development of cement concrete and its numerous uses, the question of how to obtain good and durable concrete has occupied the thought of many research workers and practicing engineers. This can be attributed to the fact that concrete has made great strides as a versatile construction material because it leads itself so well to design and fabrication. The expanded use and the demand for high quality and lasting performance of concrete have imposed corresponding demands on engineers and research workers to provide increased reliability and durability of concrete in various environmental conditions.

According to the report of ACI committee 201 (1962), durability of concrete is defined as its resistances to deteriorating influences which may through inadvertence or ignorance reside in the concrete itself, or which are inherent in the environment to which it is exposed. Thus concrete is durable when it not only maintains its structural strength and other physical characteristics but also its good appearance, its functional dimensions and wou1d not develop unsightly fissures or surface defects. Durability of concrete as a material is not primarily concerned with those more especially structural attributes or model of behavior of concrete as strength, shrinkage, creep, and elasticity, nor with cracking or any failure under load. Yet there is some interrelationship between these structural considerations and the durability of concrete as a material. It is impractical, therefore, to exclude completely all structural considerations but where these are introduced; they would be discussed only in so far as they bear on the durability of concrete as a material.

In concrete, the cement paste and cement mortar matrix play important and vulnerable roles on the ultimate strength and durability. The structural constituent of cement is such that it is very sensitive to be deteriorated when exposed to aggressive environments. In coastal areas, structures are exposed to sea water containing chlorides. Concrete storage tanks are extensively used for brine in refrigeration and desalination plants. **Table-1** shows the major ions that are found in typical open ocean sea water. Chlorides are the major ion in sea water; however, there are numerous other ions also present.

Table-1 : Average Salt Concentration in Seas (Bickzok, 1972)

Sea	g/lit	Percent
Mediterranean	38	3.8
Baltic	7	0.7
North sea and Atlantic	35	3.5
Black Sea	18	1.8
Dead Sea	53	5.3
Indian Sea	35.5	3.55

Aggressive waters containing chlorides, sulfates, CO_2, etc. can permeate concrete and react with its cementing constituents. Depending on the composition of the cement and on the quality of concrete, these chemical reactions may be accompanied by large volume changes and disruption of the concrete (Islam, M., et al 2009). Case histories of deteriorated Portland cement concrete exposed to sea water both in mild and cold climate, show that permeability is the most important characteristic determining the durability of concrete. The hydration products of Portland cement are chemically unstable to certain aggressive components present in sea water. A complete understanding of the mechanism by which salt attacks concrete in different environments would be of great importance so that guide lines could be prepared to enable more durable concrete structures to be produced.

When concrete is exposed to aggressive environments, its successful performance is dependent to a greater extent on its durability against the environment than on strength properties. An understanding of the mechanism by which the aggressive elements attack concrete is of great value to the engineer in providing a concrete to best withstand the aggression. Knowing the mechanism, a course of actions can frequently be developed for producing a concrete material which is feasible for engineering purposes.

In coastal areas and also in some places where soil contains chlorides and sulphates excessively, the concrete structures in those areas are often found to have deteriorated. Significant amount of research work has been carried out to investigate such deterioration of concrete structures exposed to similar aggressive environments (Kalousek, 1970, Mohammed, T. U, 2003, Heller, L, 1961, Suzukawa, 1980). It has been agreed that concrete is attacked more by Cl^- ions than SO_4^{-2} ions. Although there existed a considerable amount of information regarding chloride attack of concrete, it was noted that most of the investigations were concerned with the physical and mechanical properties of mature concrete due to action of deciders in general and calcium chloride in particular.

Common salt or Sodium Chloride (NaCl), is an example of a class of chemical compounds called salts. It is sufficient to regard salts as being derived from acids or bases and usually formed by reaction between them. Thus sulphuric acid will react with sodium hydroxide to form sodium sulphate, Na_2SO_4. A great many salts are soluble in water and some of them are found in nature. Sodium chloride is a principal constituent of sea water and of some naturally occurring brine. The chlorides and nitrates of ammonia, magnasium, aluminium, and iron all attack concrete, with those ammonium being the most harmful.

The deleterious effect of deicers on concrete has concerned concrete researches for several decades. However, increased use of deicers on highways and especially bridges in recent years has focused attention on the problem. The use of chemical agents, usually calcium chloride or common salt (NaCl), on exposed concrete pavements to keep them from of ice give rise to a serious problem of surface scaling commonly called “salt scaling”. Many of the factors involved in the deicer scaling problem have been discovered through research and experience. Whiteside and Sweet (1950) and Haves, J. H (1960) demonstrated that saturation of concrete is necessary for freeze-thaw deterioration. This factor has been supported by observation made by Varbeck and

Liger (1972) and Boies and Botz (1971). They showed that specimens which were in continuous contact with moisture scaled much faster than those which were permitted to dry previous to freeze-thaw cycling in the presence of deicers. It is evident, therefore, that the amount of available freezable moisture is an important variable in the deterioration of concrete by deicers in a freeze-thaw environment.

For concrete in a marine environment, there appears to be a direct correlation between low permeability, high strength, and good durability. For this reason, concrete sea structures, such as off shore platforms and harbor structures, typically are built using high quality concrete. The material recommendations call for, among other things, to water / cement ratio of 0.40 - 0.45 (by weight) and a minimum cement content of 356 kg/m^3 (600 lb/yd^3). Along with good concreting practice, this concrete should exhibit low permeability characteristics.

Permeability of concrete exposed to sea water has been found to exhibit a different behavior than that of concrete exposed to fresh water. Harrey H. Haynes (1980) found out from permeability results on concrete exposed to sea water that concrete permeated by sea water shows a decreasing permeability rate and it appears that permeability eventually stop. The actual cause for the self-water proofing behavior of concrete exposed to sea water has not been experimentally determined.

2. EXPERIMENTAL PROGRAM

The experimental work was carried out to study the strength deterioration of brick aggregate concrete in sodium chloride solution of different concentration over a period of 30 days. The variable parameters studied and the materials used were as follows:

Materials:
(a) Cement: Ordinary Portland Cement (OPC) ASTM Type-1, conforming to ASTM C-150 was used as binding material. Its physical properties and chemical compositions are given in **Table-2.**
(b) Aggregate: Locally available natural sand passing through 4.75 mm sieve and retained on 0.075 mm sieve was used as fine aggregate. The coarse aggregate was bricks chips with a nominal size of 20 mm.

Table-2: Physical properties and Chemical composition of OPC

Sl. No.	Characteristics	Value
1.	Blaine's Specific Surface (cm^2/gm)	2900
2.	Normal Consistency	26%
3.	Soundness by Le Chatelier's Test (mm)	4.5
4.	Specific Gravity	3.15
5.	Setting Time (a) Initial (min) (b) Final (min)	 70 175
6.	Compressive Strength (a) 3 days (MPa) (b) 7 days (MPa) (c) 28 days (MPa)	 15.2 20.2 30.4
7.	Calcium Oxide (CaO)	64%
8.	Silicon Dioxide (SiO_2)	21%
9.	Aluminum Oxide (Al_2O_3)	6%
10.	Feric Oxide (Fe_2O_3)	3.5%
11.	Magnesium Oxide (MgO)	1.2%
12.	Sulfer Trioxide (SO_3)	2.5%
13.	Loss on ignition	1.2%
14.	Insoluble matter	0.6%

Variables:
(a) Curing Solution: Plain water (PW) as well as artificially made chloride environments (3%, 5% and 10% NaCl solution) were used for curing the test specimens. Typical SW is found to contain around 2.7% NaCl and to simulate this effect, three different concentrations ie. 3%, 5% and 10% NaCl solutions were used. PW curing environment was used as controlled condition. Enhanced salt concentrations were used to get accelerated effect within short span of time.

(b) Concrete Quality: Concrete having mix ratio 1:2:4 with three different w/c ratios ie. 0.45, 0.50, 0.55 were used.
(c) Exposure Period: Concrete specimens were tested periodically after the specified curing periods of 7, 14 and 28 days in plain water, 3%, 5% and 10% NaCl environments.
(d) Size of Specimens: 150 mm dia × 300 m high cylindrical specimens were prepared following ASTM standard procedure.
(e) Casting and Curing: A total of 216 Nos. concrete test specimens were cast in the laboratory and then kept at 27^0C temperature and 90% relative humidity for 24 hours. After demoulding, all the specimens were exposed to plain water and other chloride environment at ambient temperature. After specific curing, the specimens were taken out for conducting tension and compression strength test. The test data were analyzed critically and presented in graphical form so as to provide some useful conclusion.

3. RESULT AND DISCUSSIONS

As stated in the test program, a total of 216 concrete test cylinders were cast of which 108 specimens were for compressive strength and the rest for tensile strength test. The variation of compressive strength with ages for different water / cement ratio and cured both normally and in salt solutions of different concentrations are presented in **Fig.1 to Fig.3**. The compressive strength of concrete cured in sodium chloride solutions followed the same nature as that of the normally cured concrete with the subsequent reduction of strength. More the salt concentration, the higher was the strength reduction although the rate was not proportional. Also it was noticed that the salt environments had significant influence on the rate of gain of strength of concrete with age. For example, for water / cement ratio 0.45, the f’c 28/f’c 7 was equal to 1.92 for normally cured concrete whereas the same ratio was 1.80 for the concrete cured in 10% salt solutions. The same ratio was 1.91 and 1.81 for 3% and 5% salt solutions respectively.

The rate of gain of strength was also found to vary with different water/ cement ratio. For w/c ratio: 0.5 and 0.55, f'c 28/f'c 7 was equal to 1.84 and 1.75 for normally cured specimens and 1.67 and 1.52 respectively for concrete specimen cured in 10% salt solutions. It may be argued from the above discussion that the rate of gain of strength of salt affected concrete is 5-10% less than that of normally cured concrete and its amount is found to vary with different water/cement ratio.

The results of tensile strength test of concrete with different water/cement ratios subjected to the same curing environments are plotted in **Fig.4 to Fig.6.** The behavior of tensile strength was more or less similar in nature as that of the compressive strength showing the reduction of strength with the increase of salt concentrations. However, the gain of tensile strength of salt affected concrete is observed to be similar to that of normally cured concrete.

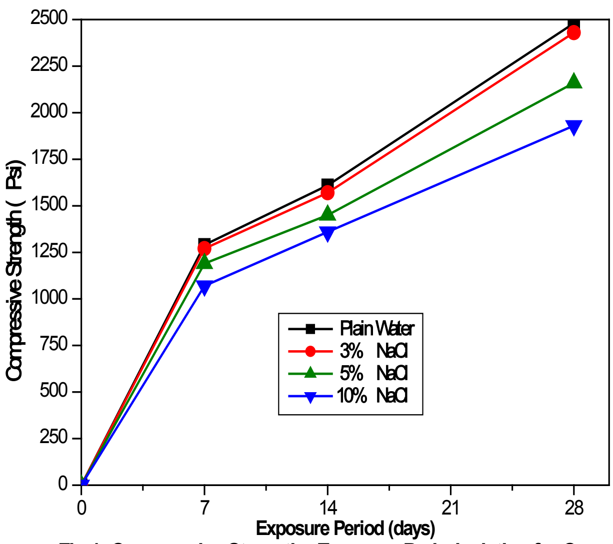

Fig.1: Compressive Strength - Exposure Period relation for Concrete Exposed to NaCl solution (w/c = 0.45)

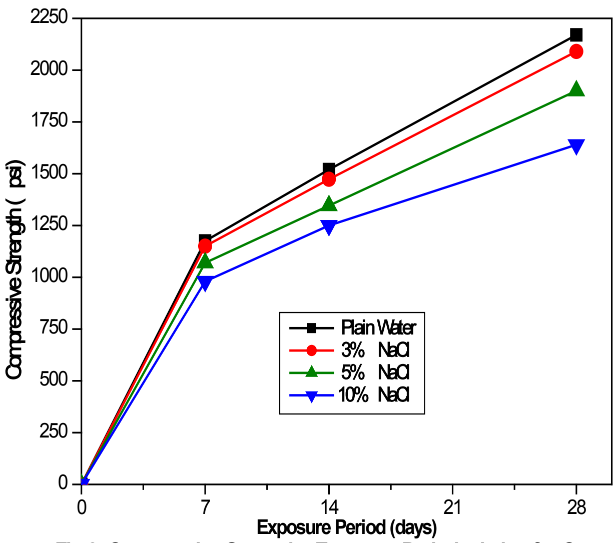

Fig.2: Compressive Strength - Exposure Period relation for Concrete Exposed to NaCl solution (w/c = 0.5)

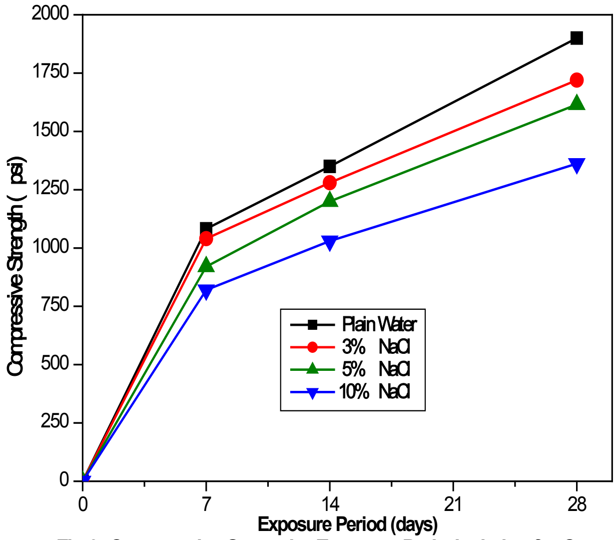

Fig.3: Compressive Strength - Exposure Period relation for Concrete Exposed to NaCl solution (w/c = 0.55)

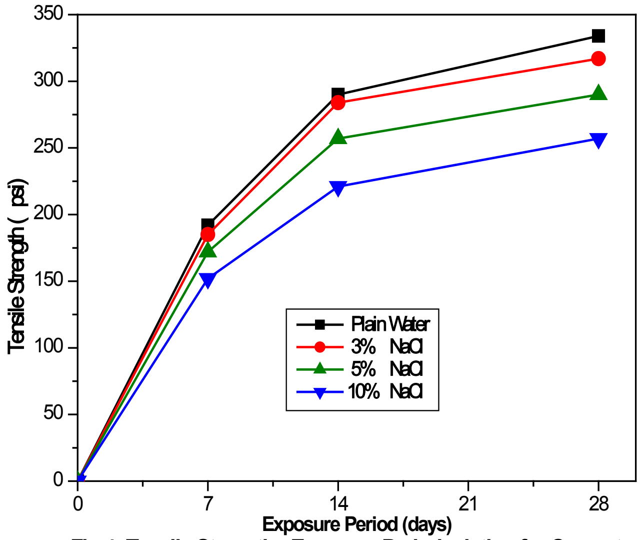

Fig.4: Tensile Strength - Exposure Period relation for Concrete Exposed to NaCl solution (w/c = 0.45)

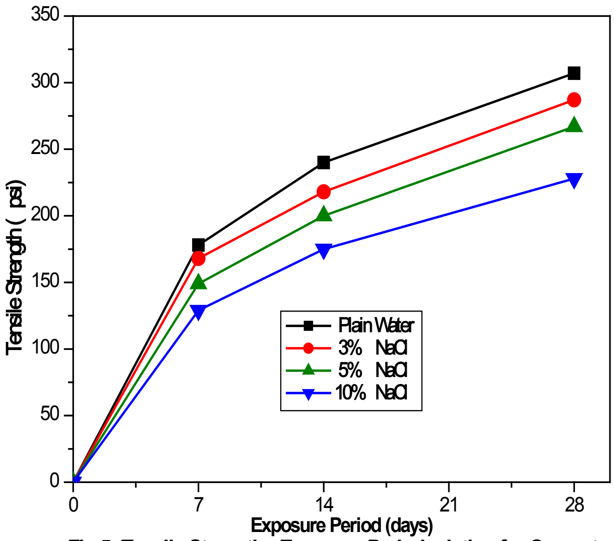

Fig.5: Tensile Strength - Exposure Period relation for Concrete Exposed to NaCl solution (w/c = 0.5)

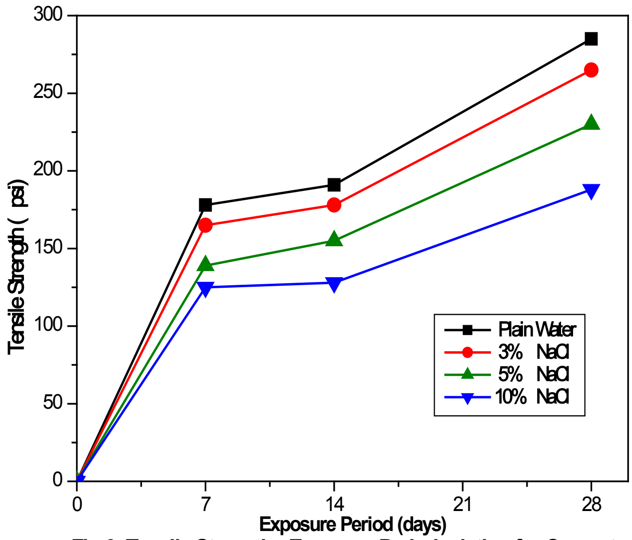

Fig.6: Tensile Strength - Exposure Period relation for Concrete Exposed to NaCl solution (w/c = 0.55)

For a particular curing period, the reduction of strength for different curing conditions were recorded to be more with the gradual increase of water/cement ratio both for compressive and tensile strength of concrete. Again for a given water/cement ratio, strength deterioration of concrete in different curing conditions was found to increase with the increase of curing periods. The effect of water / cement ratio on the strength of concrete (both for tensile and compressive) subjected to salt solutions are discussed with the help of experimental results plotted in **Fig.7 to Fig.12.** For compressive strength test, the normally cured concrete was found to obey the Duff. Abrams law and the plotted curve were concave upward. But concrete cured in salt solutions showed different behaviors as compared to normally cured concrete.

For tensile strength test, both normally and specially cured concrete behaved in the similar manner and the corresponding plotted curves followed the nature of the standard one especially in early stages of hydration. Some deviations were observed as curing periods increased. With the increase of curing period, the curvature of the strength vs. water/cement ratio curve was found to decrease and become nearly straight at 28 day of curing period. Again, this behavior was seen to be accelerated with the increase of salt concentrations.

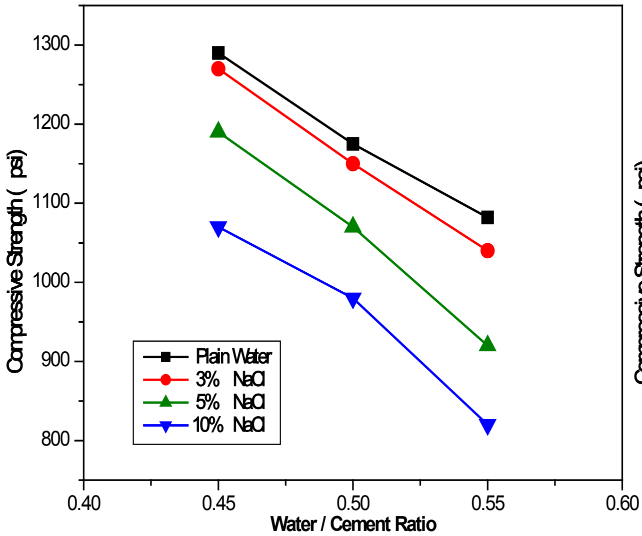

Fig.7: Compressive Strength - Water Cement ratio relation for Concrete Exposed to NaCl solution (7 days curing period)

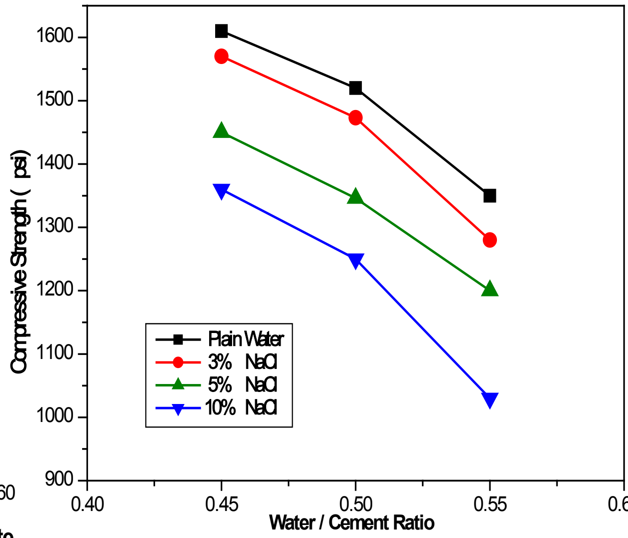

Fig.8: Compressive Strength - Water Cement ratio relation for Concrete Exposed to NaCl solution (14 days curing period)

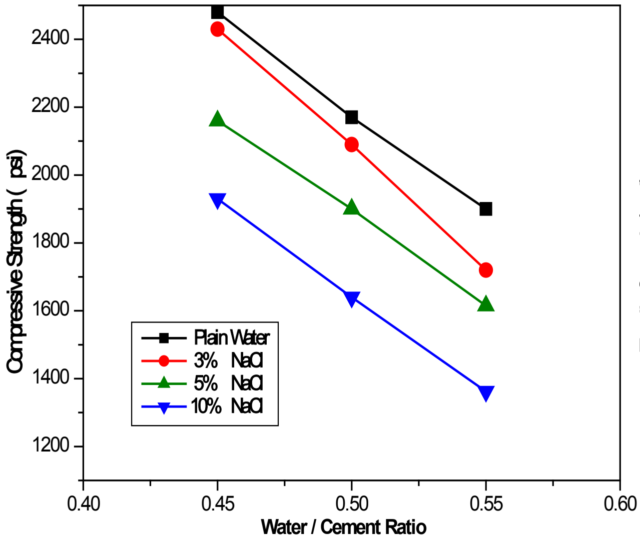

Fig.9: Compressive Strength - Water Cement ratio relation for Concrete Exposed to NaCl solution (28 days curing period)

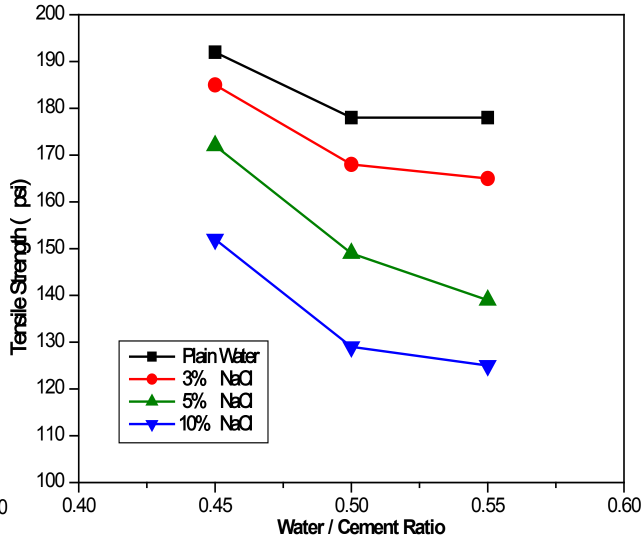

Fig.10: Tensile Strength - Water Cement ratio relation for Concrete Exposed to NaCl solution (7 days curing period)

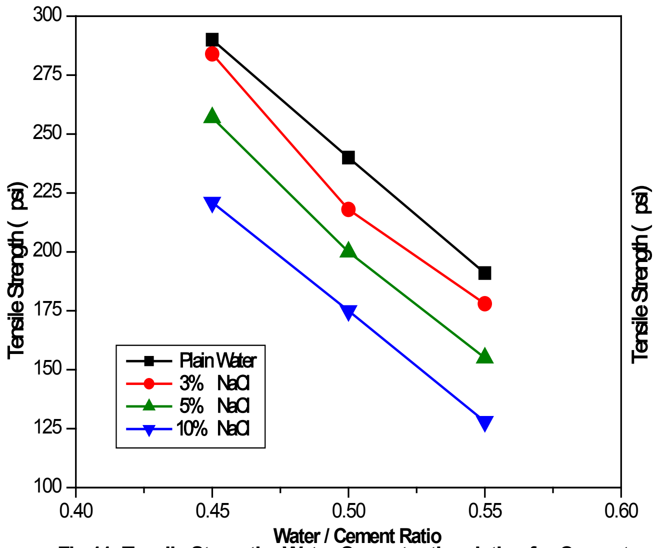

Fig.11: Tensile Strength - Water Cement ratio relation for Concrete Exposed to NaCl solution (14 days curing period)

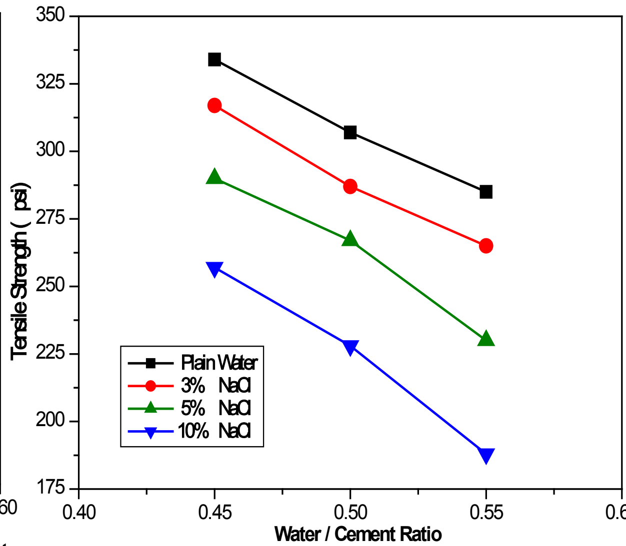

Fig.12: Tensile Strength - Water Cement ratio relation for Concrete Exposed to NaCl solution (28 days curing period)

Fig.13 to Fig.15 revealed the percent reduction of compressive strength with the curing conditions of the test specimen for different curing periods and water/cement ratio. The strength reduction rate was found to increase gradually both with the increase in salt concentration and curing period. For water/cement ratio 0.45, the 7 days curing of specimen experienced more reduction of strength in comparison to 14 days curing period specially in 10% solution though in 3% and 5% solution, it followed the general rule. At 28 days curing period, the strength loss was maximum and the value was about 22%. At water / cement ratio 0.55, for higher salt concentration that reduction for 7 days curing was reported to be higher in comparison to 14 days curing. Again the maximum strength reduction was recorded to 28% in 10% salt solutions. It was also noticed that with the increase of water/cement ratio, the strength reduction rate increased specially for 28 days curing period.

The percent reduction of tensile strength of concrete subjected to different curing condition and water/cement ratio are explained graphically through the **Fig.16 to Fig.18.** Unlike compressive strength, the reduction of tensile strength was found to be maximum for 14 day curing periods in many of the cases of different salt concentration and water/cement ratio, the maximum value being 34% at 28 days for water/cement ratio 0.55 and 10% salt concentration. Here also the strength reduction rate accelerated with the increase of salt concentration and water/cement ratio like compressive strength pattern but the rate decreased gradually as curing period was exceeded by 14 days. However, for all curing periods and curing conditions, the deterioration regarding tensile strength was more possibly due to the formation of micro cracks and their subsequent propagation.

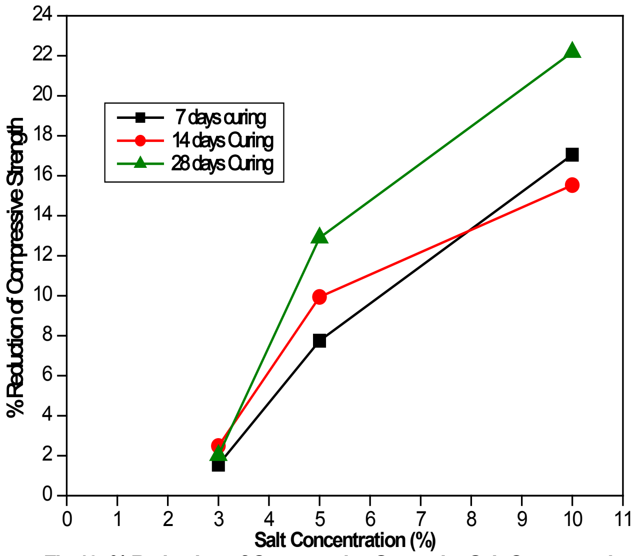

Fig.13: % Reduction of Compressive Strength - Salt Concentration relation for Concrete (w/c ratio 0.45)

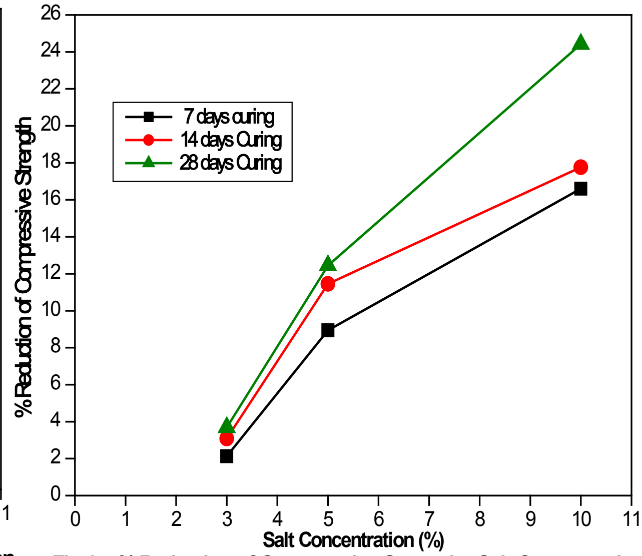

Fig.14: % Reduction of Compressive Strength - Salt Concentration relation for Concrete (w/c ratio 0.5)

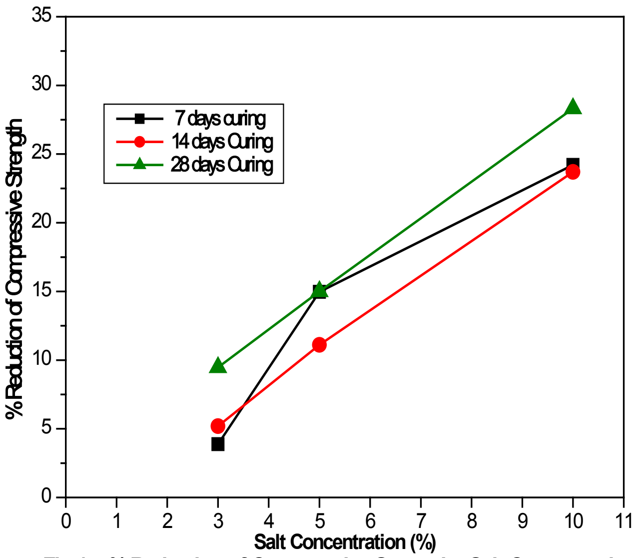

Fig.15: % Reduction of Compressive Strength - Salt Concentration relation for Concrete (w/c ratio 0.55)

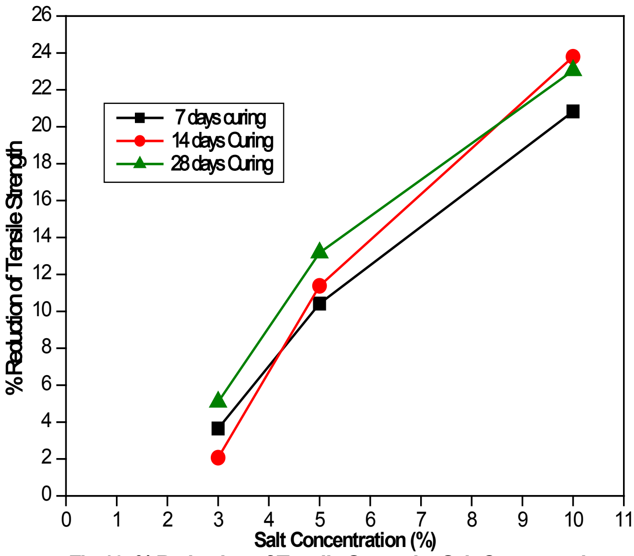

Fig.16: % Reduction of Tensile Strength - Salt Concentration relation for Concrete (w/c ratio 0.45)

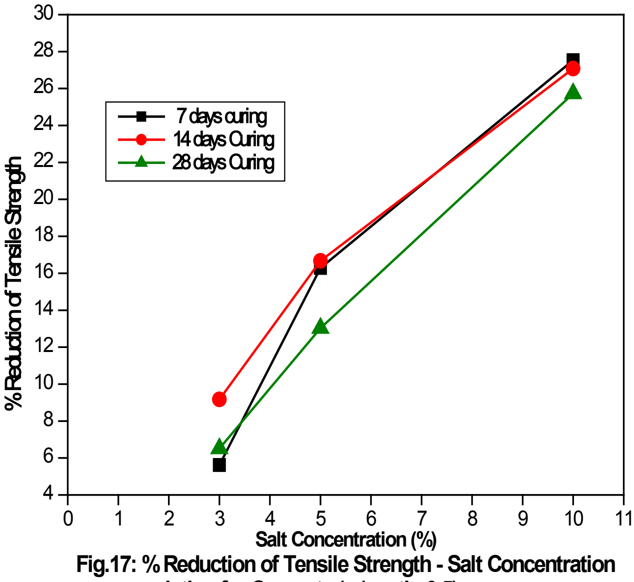

Fig.17: % Reduction of Tensile Strength - Salt Concentration relation for Concrete (w/c ratio 0.5)

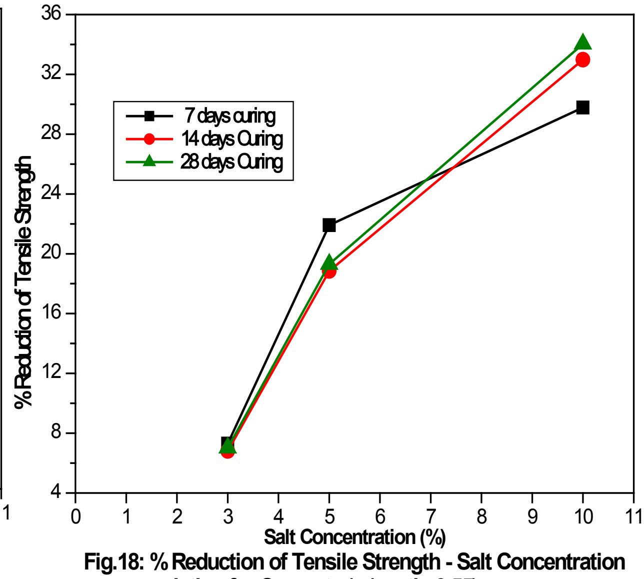

Fig.18: % Reduction of Tensile Strength - Salt Concentration relation for Concrete (w/c ratio 0.55)

Thus it is seen that sodium chloride environment has significant effect on the strength gain of concrete. It is also noticed that this effect increases accelerately with the increase of salt concentration. For low water/cement ratio i.e. in dense or compacted concrete, the salt attack is not so prominent in comparison to weak or porous concrete. Many workers have tried to explain the causes of deterioration regarding strength of concrete subjected to sodium chloride environments in various ways.

The possible reason for such deterioration is that in porous or week concrete, the aggressive salt ions penetrate into the interior of the mass and change the hydration mechanism with the formation of some expansive products such as ettringite (calcium sulfoaluminate) or its various forms according to Bickzok (1972). As a result, micro cracks are developed within the concrete mass thereby weakening the interlocking capacities of the hydrated products with the aggregate particles.

5. CONCLUSIONS

On the basis of results obtained from the testing of 216 cylindrical test specimens of brick aggregate concrete cured both normally and in sodium chloride solutions of different concentration, the following conclusions are drawn.

(a) Concrete specimens cured in sodium chloride solutions of different concentration showed the reduction of both compressive and tensile strength as compared to the plain water cured concrete. More the salt concentration of the curing environments; the higher was the strength reduction rate although it was not proportional.
(b) Strength deterioration of concrete was found to be the function of both water/cement ration and curing period. For a particular curing period, the strength reduction was found to increase with the gradual increase of water/cement ratio for both compressive and tensile strength of concrete. Also the strength deterioration of concrete for different curing condition was found to increase with the increase of curing periods.
(c) Concrete cured in salt solutions behaved in dissimilar fashion in regard to its strength variation with water/cement ratio in comparison to the normally cured concrete. Although both salt affected and normally cured concrete showed the gradual reduction of strength with the increase of water/cement ratio; their variational patterns were not similar. The overall strength deterioration of the concrete specimens was observed to vary in the range of 3 to 28% for compressive strength and 5 to 34 % for tensile strength.
(d) The gain of compressive strength of salt affected concrete is around 5 to 10% lower than that of normally cured concrete and its amount is found to vary with different w/c ratio. However, the gain of tensile strength was almost similar to normally cured concrete.
(e) As per related literature, the possible causes for strength determination of concrete specimens might be due to the dissolution of the compounds rich in lime and the formation of expansive ettringite. Sodium chloride of higher concentrations produced calcium chloride by reacting with calcium hydroxide hydrates that leached out

resulting in the formation of larger pores inside the specimens. The penetrated chloride ions changed the normal hydration mechanism with the formation of some expansive products such as calcium chloroaluminate, calcium sulfoaluminate (ettringite) etc. which developed micro cracks that caused such determination.

6. REFERENCES

ACI Committee 201. (1962). "Durability of Concrete in Service". ACI Journal Proceedings. Vol.59, No.12, 1771-1820.

Biczok, I. (1972). Concrete Corrosion, Concrete Protection, Akademiai Kiado, Budapest.

Boies, D. B., and Bortz, S. (1971). "Economical and Effective Deicing Agents for use on Highway Structure". NCHRP Report-19.

Harrey H. Haynes. (1980). "Permeability of Concrete in Sea Water". ACI Publication, SP-65, 21-31.

Haves, J. H. (1960). "Thermal Analysis of the Freeze-thaw Mechanism in Concrete". Bulletin-59. Engineering Research, University of Keutueky, Lexington.

Heller, L. & Ben-Yair, M. (1961). "Formation of Calcium Choraluminate by interaction of Portland Cement with sea-water and chloride solution". Nature, Vol.191. No.4787, 488-489.

Islam, M. M. Islam, M. S. Mondal, B. C. and Das, A.(2009). "Strength Behavior of Mortar Using Slag with Cement in Sea Water Environment". Journal of Civil Engineering, IEB, Vol.37, No.2, 111-122.

Kalousek, G. L. & Benton, E. J. (1970). "Mechanism of Sea Water Attack on Cement Past". Journal of American Concrete Institute. Vol.67, 187-192.

Mehta, P. K. (1980). "Durability of Concrete in marine environment-A review". Performance of Concrete in Marine Environment. ACI Publisher. SP-65. 1-20.

Mohammed, T. U. & Hamida, H. (2003). " Durability of Concrete made with different Water-reducing Chemical Admixtures in Tidal Environment". ACI Material Journal, Vol.100. No.3. 194-202.

Suzukawa, Y. (1980). "Resistance of Hardened Cement Mortar to Magnesium Chloride Solution". Seventh International Symposium on Chemistry of Cement. Paris, 124.

Verbeck G. J. and Klieger, P. (1972). "Studies of Salt Sealing of Concrete". HRB Bulletin-150.1-17.

Whileside, T. M. and Sweet, H. S. (1950). "Effect of Mortar Saturation in Concrete Freezing and Thawing Tests". Proceedings, HRB, Vol.30. 204-211.

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PERFORMANCE OF FLY ASH CONCRETE IN SEAWATER AGAINST FREEZE-THAW ACTION

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ABSTRACT

Concrete deterioration from freeze/thaw cycles has been a major problem in cooler areas of many countries. It begins when water enters into the voids inside the concrete. Leaching of calcium hydroxide, product of hydration of portland cement, provides greater voids for water to occupy, thereby aggravating the rate of deterioration. Concrete subjected to freezing and thawing can be damaged externally or internally. High quality fly ash combines with calcium hydroxide to produce additional cementitious materials, thereby reducing the amount of leachable calcium hydroxide. As a result, permeability and porosity are reduced. Fly ash reduces the amount of water required in the mix by approximately 2% to 10%, because the spherical shape of the fly ash particles reduces bleed channels and void spaces. This paper presents a part of an experimental study on the freeze–thaw effect of concrete specimens exposed to artificial seawater simulating marine environment and plain water over 180 cycles. Two different grades of concrete M38 and M28, each with four different cement replacement levels (20, 30, 40 and 60%) were used for the experimental program. Ordinary Portland cement (OPC) concrete was also used as reference concrete. The deteriorative effects on concretes were measured by studying weight and volume change, compressive strength characteristics of the deteriorated test specimens. Among all the concretes studied, the optimum amount of cement replacement is reported to be around 30 to 40%. The study reveals that fly ash concrete has better resistance against freeze-thaw deterioration due to its pozzolanic activity that creates more calcium silicate hydrate (CSH) gel and fills capillaries and bleed water channels occupied by water soluble lime.

Keywords: Compressive strength, Durability, Fly ash, Freeze-thaw, Seawater*s*

1. INTRODUCTION

Concrete deterioration due to freeze-thaw cyclic action has been a major problem in cooler areas of different countries. Freeze-thaw deterioration begins when water enters into the voids of concrete. Leaching of calcium hydroxide, which is produced during the hydration of portland cement, provides greater voids for water to occupy, thereby aggravating the rate of deterioration. Freezing of water or salt solution in the concrete pores may cause severe deterioration and considerable reduction of service life. It is commonly known that plain water freezes at 0°C under normal atmospheric pressure. When water freezes, the volume increases by 9% and as water turns to ice, high pressure is generated in the adjacent concrete (Hale, 2009). For reinforced concrete structures, other than the degradation of concrete materials, the existence of cracks under service loads is believed to be one of the main causes of deterioration of reinforced concrete structures serviced in cold marine environment or snow melting conditions (Diao, 2011). Chloride or other chemical ions would penetrate into concrete through cracks and induce corrosion of reinforcing steel bars. The amount of permeation is determined by the thickness of the concrete cover and the width of the cracks (Djerbi, 2008). To simulate the working conditions of reinforced concrete structures in cold coastal regions, the presence of structural loads in addition to the harsh environmental factors were considered in a number of recent studies (Diao, 2011).

Concrete has been extensively used as the basic construction material for various types of offshore/onshore structure over the several decades. Thus structural concrete in cold coastal regions are exposed to coupling effects of freeze-thaw cycles and seawater corrosion. Further-more, because of the wide use of deicing salt for melting ice on the roads, salt is mostly available near these structures. Besides creating pressure through osmosis and crystallization, deicing chemicals increase the degree of concrete saturation and keep concrete pores at or near maximum fluid saturation, thus increasing the risk of frost damage (Litvan, 1976). The salt in deicing solution also decreases the freezing point of concrete pore solution, leading to significant hydraulic pressure (Setzer, 1976). Because of the salt concentration varies with the distance from the exposed concrete surfaces, various amounts of ice may form in different layers under the concrete surface resulting in deformation of the layers and generation of pressure between the layers.

One of the advances in concrete technology is the development of fly ash concrete and its use in it. Fly ash is a by-product from combustion of pulverized coal. As the coal is heated to high temperatures, it liquefies. It is thereafter cooled rapidly, which forms spherical particles. The fly ash consists mainly of silica (SiO_2), aluminium oxide (Al_2O_3), iron oxide (Fe_2O_3) and calcium oxide (CaO). Fly ash combines with calcium hydroxide to produce additional cementitious materials, thereby reducing the amount of calcium hydroxide that may be leached out of the concrete. Leaching of the calcium hydroxide increases concrete voids which can accelerate freeze-thaw damage. As a result, permeability and porosity are reduced. Fly ash also fills the minute voids creating a denser and less absorptive concrete. It reduces the amount of water required in the mix by approximately 2% to 10%, because the spherical shape of the fly ash particles reduces bleed channels and void spaces. Reduction of bleed channels limits the entrance of water; fewer void spaces mean less space for water to accumulate.

Chloride ions from sea water and de-icing salts can penetrate into concrete by transport of chlorides in water, diffusion of the ions in water and by absorption. If the chloride ions reach the reinforcement, corrosion may occur. The penetration of chloride ions is dependent on the permeability of the concrete; a more permeable concrete will lead to less resistance against penetration (Neville, 2003). Concrete with fly ash has shown better resistance against chloride penetration than concrete with ordinary Portland cement. The active alumina (Al_2O_3), which exists in larger amounts in fly ash than in Portland cement, is able to bind the chloride ions. According to Dhir (1999), the binding capacity was found to be at maximum at a replacement of fly ash of 50% of the cement, but optimum at about 30%. Concrete with fly ash replacing 33% of the cement, the binding capacity was four times larger than for ordinary Portland cement. Furthermore, the binding capacity increased with the concentrations of chloride ions. Replacement of the cement by 30% fly ash was found to improve the resistance against chloride ions with two to four times. A more matured concrete will be less permeable, thus will have more resistance against chloride ingress.

The key to prevention of sulfate attack is to tie up the free lime and calcium aluminates to eliminate the possibility of ongoing reactions. Increased sulfate resistance of concrete containing fly ash may be explained by the reaction of silica, alumina and ferric oxide found in fly ash with calcium hydroxide liberated during the hydration of Portland cement to form relatively stable cementitious compounds. Greater impermeability of such type of concrete reduces penetration of sulfate solutions and results in improved resistance to sulfate attack. Fly ash not only reduces the permeability of the concrete, but because of reaction of these materials with available alkalis, it removes that essential component required for Alkali Aggregate reaction (AAR) and thus it is an effective means of reduction the risk of AAR occurring.

Drahushak-Crow and von Fay (1991) studied freezing and thawing durability of concretes made with three different fly ashes. Fly ash concrete mixtures were proportioned for five different cement replacement levels (10, 30, 50, 75, 100 %) with fly ash. The number of cycles to failure depended greatly upon type of fly ash, amount of cementitious content, and type of curing.

Penttala (2006) studied the surface and internal deterioration of concrete exposed to saline and nonsaline freeze-thaw loads. The results indicated that internal damage determines the need for air entrainment in high-strength concretes, whereas in normal or low-strength concretes, surface scaling determines the need for higher air content compared with the internal damage by freeze-thaw mechanism.

Achintya and Prasad (2003) studied the behavior of concrete in the freeze-thaw environment of seawater. In this study, concrete samples were completely submerged in plain water and simulated seawater, and also in a non submerged state. This study showed that specimens subjected to seawater underwent change in size and shape along with substantial abrasion, erosion, and crumbling on the surface, whereas the exposed surfaces of samples became uneven when subjected to plain water and the atmospheric condition.

It is important to have detail information regarding the concrete durability on freezing and thawing in sea water in view of the increasing use of concrete in the sub-arctic/arctic region. The freeze-thaw action on structural concrete in the splash/tidal zone has its own characteristics and is dependent on ambient air and sea water temperature. In addition, the chemical attack of the sea water on the cement constituents is found to lead to a more pronounced deterioration in the concrete structure. The major aim of this work is to evaluate the freezing and thawing durability of concrete made with Class F fly ash. The properties evaluated were the weight change, volume change and compressive strength of concrete with or without fly ash. The results of this investigation would provide data for establishing appropriate mix proportions for fly ash concretes to be exposed in freezing and thawing environment and in sea water.

2. EXPERIMENTAL PROGRAM

The experimental program was planned to study the suitability of using Boropukuria fly ash as partial replacement of cement in making structural concrete exposed to Freezing-Thawing environment in plain as well as sea water.

2.1 Materials Used

(a) Cement: ASTM Type-I Ordinary Portland Cement (OPC) was used as binding material. Chemical compositions of OPC are given in **Table-1.**
(b) Fly ash: A low calcium fly ash compiling with ASTM Class F Fly ash collected from Baropukuria, Bangladesh was used as supplementary cementitious material. Chemical analysis of the fly ash conducted using X-ray fluorescence (XRF) study is also shown in **Table-1.**
(c) Aggregate: Locally available natural sand passing through 4.75 mm sieve and retained on 0.075 mm sieve with fineness modulus 2.58 and specific gravity 2.61 was used as fine aggregate. The crushed stone with a maximum nominal size of 12.5 mm, fineness modulus 6.58 and specific gravity 2.70 was used as coarse aggregate.

Table-1 : Chemical Composition (%) of Ordinary Portland Cement and Fly Ash

Constituents	Composition	OPC	FA
Calcium Oxide	CaO	65.18	0.65
Silicon Di-Oxide	SiO_2	20.80	51.49
Aluminum Oxide	Al_2O_3	5.22	31.60
Ferric Oxide	Fe_2O_3	3.15	2.80
Magnesium Oxide	MgO	1.16	0.28
Sulfur Tri-Oxide	SO_3	2.19	0.19
Sodium Oxide	Na_2O	--	0.18
Loss on Ignition	--	1.70	4.2
Insoluble Residue	--	0.6	--

-- = not measured items.

2.2 Variables Studied

(a) Curing Water: Plain water (PW) as well as sea water (SW) were used for curing the specimens. SW was artificially made by mixing tap water with exact amount of principal salts found in natural sea water. The composition of artificial sea water is given in **Table-2 (**Mayers, 1969)**.**

Table-2: Specified Salt Contents of Artificial SW Used in Experimental Program [29]

SALT	Chemical Formula	Amount (gm)	Remarks
Sodium Chloride	$NaCl$	27.2	These amounts of salts were dissolved in plain water to prepare 1000 gm of Sea water
Magnesium Chloride	$MgCl_2$	3.8	
Magnesium Sulfate	$MgSO_4$	1.7	
Calcium Sulfate	$CaSO_4$	1.2	
Potassium Sulfate	K_2SO_4	0.9	
Calcium Carbonate	$CaCO_3$	0.1	
Magnesium Bromide	$MgBr_2$	0.1	
Total		**35.00**	

(b) Exposure Condition: Freezing-Thawing arrangement was created in a freeze-thaw chamber. In each freeze-thaw cycle, the temperature was varied from (-17.8^{0}C) to (+4.4^{0}C) over a total period of 24 hours (8+4 hours for freezing and thawing; 7+5 hours kept at two terminal temperatures) **(Refer to Plate No.1)**.
(c) Exposure Periods: 30, 90 and 180 freeze-thaw cycles was used as exposure period after 28 days of precuring in plain water.

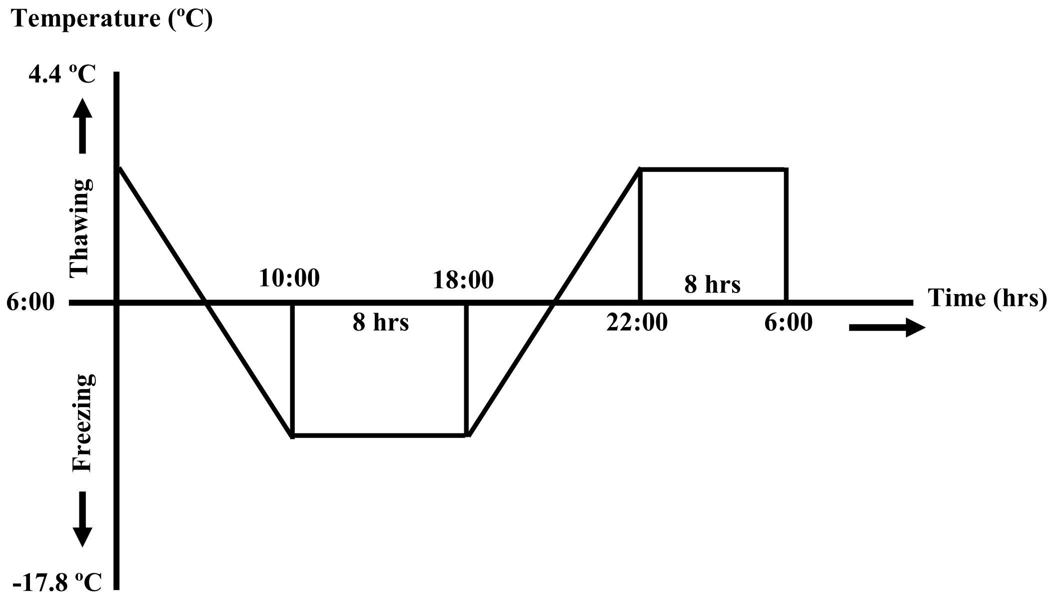

Plate No.1: Schematic Freeze-Thaw Cycle

2.3 Mix Design and Sample Preparation

Two different grades of concrete namely M38 and M28 were used in the program. Four different mix proportions of cement fly ash (80:20, 70:30, 60:40 and 40:60) were used as cementitious material. Cement fly ash mix ratio of 100:0 i.e. plain concrete specimens were also cast as reference concrete for comparing the properties of fly ash concrete. Relevant information of different concrete mixes is given in **Table-3.**

Table-3 : Mix Proportions and Properties of Fresh Concrete

Mixture constituent & properties	Grade of Concrete	
	M38	M28
Cement (kg/m^3)	500	435
Water (kg/m^3)	218	218
Sand (kg/m^3)	520	545
Stone Chips (kg/m^3)	1120	1150
water/cement Ratio	0.44	0.50
Slump (mm)	60	68
Air content %	1.1	1.3

2.4 Size of Specimens

A total of 300 nos of cubical specimens of 100 mm size from five different types of fly ash concretes were cast as per requirement for conducting strength test. The small size of specimen i.e. 100 mm cube was selected in order to accommodate large number of specimens in the limited sized curing tanks. The specimens were demoulded after 24 hours of casting and cured in plain water at 27±2°C. Concrete specimens were designated as per grade of concrete and amount of fly ash as a percentage of total cementitious material. Thus M38FA40 concrete means grade of concrete is M38 and cement fly ash mix ratio is 60:40.

3. EXPERIMENTAL PROCEDURES

3.1 Strength

The concrete specimens were tested for compressive strength at 30, 90 and 180 freeze-thaw cycles in accordance with the BS EN 12390-3:2009. The reported strength in each case is taken as the average of three tests results.

3.2 Freezing-Thawing Test

In this study, as per ASTM C 666, the Procedure A was used and according to this procedure, the temperature of the curing water condition concrete specimens was lowered from 4 to -17.8 °C and raised it from -17.8 to 4 °C in 4 hours.

4. RESULTS AND DISCUSSIONS

Concrete test specimens exposed in the sea water and plain water are taken out after specific cycles of freeze-thaw, for conducting various tests. The test results are graphically presented and discussed in the following sections:

4.1 Visual Examination

The concrete specimens subjected upto 180 cycles of freeze-thaw actions in the submerged state of plain water and sea water are shown in **Plate No.2 to plate No.7.** After visual inspection, it is seen that concrete specimens in sea water have lost their dimensional stability with substantial erosion and splitting/crumbling on the surface whereas in plain water, showed marginal erosion and concrete surface tend to become uneven. Some changes in color from the original lime gray to reddish gray of the specimens in sea water have also been observed which indicates either the salts deposition on the concrete surfaces or leaching out of portlandite, $Ca(OH)_2$.

Plate No.2: FA Concrete specimens after 30 F-T cycles in PW

Plate No.3: FA Concrete specimens after 30 F-T cycles in SW

Plate No.4: FA Concrete specimens after 90 F-T cycles in PW

Plate No.4: FA Concrete specimens after 90 F-T cycles in SW

Plate No.6: FA Concrete specimens after 180 F-T cycles in PW

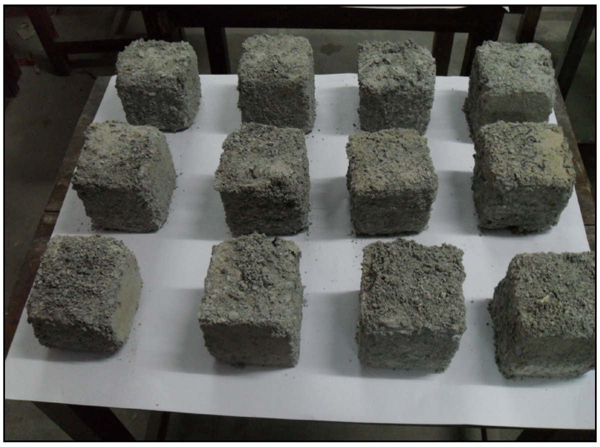

Plate No.7: FA Concrete specimens after 180 F-T cycles in SW

4.2 Weight Change

The change in weight of various concrete specimens of grade M38 and M28 in different exposure conditions and for various freeze-thaw cycles have been illustrated in **Figure.1** and **Figure.2** respectively. A close examination reveals that at the end of 30 cycles of freeze-thaw, the specimens in sea water exhibit a higher percentage (nearly 1.1 to 2.6%) of weight gain as compared to the plain water cured specimens (nearly 0.9 to 2.2%). This increase in weight may be primarily due to the ingress of sea water or plain water into the concrete. But after longer period of curing, a significant difference in the trend of weight change for the concrete specimens exposed to sea water has been found as compared to that for the specimens placed in plain water.

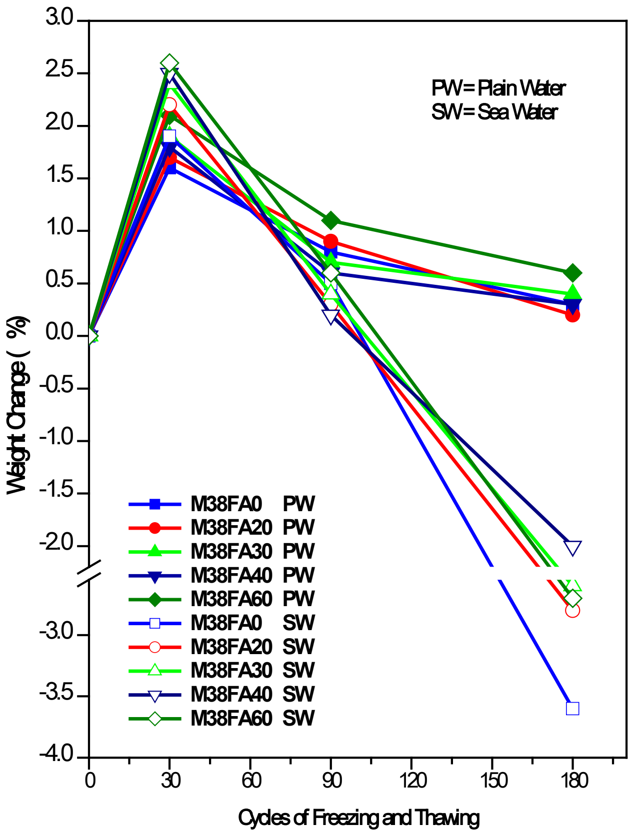

Fig.1: Weight Change - Freeze Thaw Relation for M38 Grade Concrete

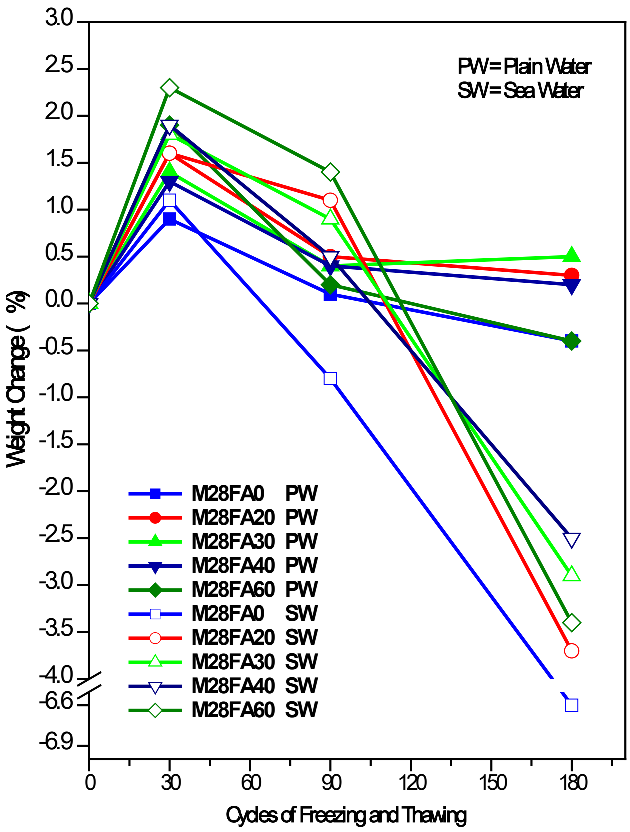

Fig.2: Weight Change - Freeze Thaw Relation for M28 Grade Concrete

At the end of 90 cycles of curing, plain water cured concrete specimens exhibit higher percentage of weight gain as compared to sea water cured concrete. After 180 cycles of freeze-thaw, a considerable change (loss) in the weight lying between -0.4 and 0.6% is observed for the concretes exposed to the plain water environment whereas for the similar specimens placed in sea water, this change is found to lie in the range of -2 to -6.6%. The higher loss in weight of the specimens exposed to sea water is primarily due to crumbling of outer surfaces of the specimens caused by crystallization of sea salts in the voids of concrete and their subsequent expansion during freeze-thaw cycles. From the figure it is clear that fly ash concrete shows better resistance against excessive weight change value as compared to plain concrete particularly after long freeze-thaw cycle. It may be due to the development of its resistance to water/salt ions penetration inside concrete as the rate of hydration of fly ash concrete in relatively slow. Also it is seen that, higher grade concrete has better resistance against weight change compared to lower grade concrete in freeze-thaw action.

4.3 Volume Change

Volume change of fly ash concrete of two different grades of concrete M38 and M28 exposed to sea water and plain water for different period of freeze-thaw cycle are illustrated in **Figure.3 and Figure.4** respectively. It is evident from these figures that the volume change of concrete specimens is relatively higher when cured in sea water as compared to plain water under Freeze-Thaw action. At the initial stage of curing, i.e. after 30 cycles of freeze-thaw, volume of all the specimens are observed to be increased. It may due to hydration reaction of cement in presence of ingress of sea or plain water inside the concrete mass. At the end of 90 cycles of freeze thaw, sea water cured concrete specimens have exhibited volumetric change of nearly -0.2 to -0.53%, whereas a volume change of -0.19 to 0.1% has been found in the specimens placed in the plain water. The volume change of the concrete specimens under freeze thaw cycle has been found due to surface erosion and splitting. Also for longer period of curing, the effect of sea water is devastating as compared to plain water. At the end of 180 cycles of Freeze-thaw, volume change value is observed to lie between -0.05 to -0.21% for plain water and -0.36 to -0.83% for sea water cured concrete. The decrease in volume resulting from erosion/crumbling of outer surfaces of concrete may be attributed to the deposition of chemical compounds into the voids of concrete, the crystallization as well as their expansion due to freezing of the entrapped water inside the voids. Also from the figures it is clear that fly ash concrete shows much more resistance against volume change as compared to OPC concrete for relatively higher cycles of freeze-thaw.

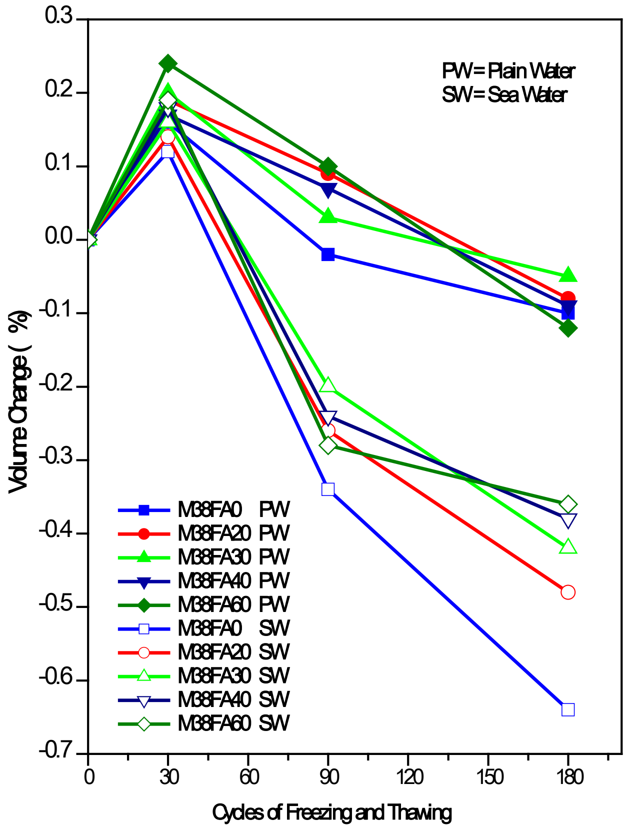

Fig.3: Volume Change - Freeze Thaw Relation for M38 Grade Concrete

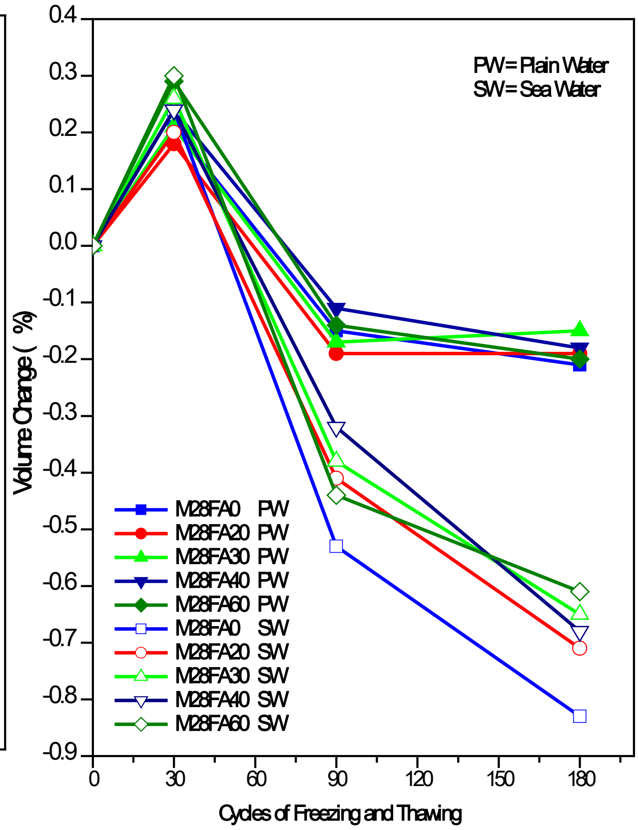

Fig.4: Volume Change - Freeze Thaw Relation for M28 Grade Concrete

4.4 Compressive Strength

Compressive strength of OPC and fly ash concrete of M38 and M28 grades has been graphically presented in **Figure.5 and Figure.6**. Also for the ease of comparison, the relative compressive strengths at different freeze-thaw cycles and in different curing water are plotted in **Figure.7 and Figure.8**. The strength values corresponding to '0' cycle curing age represent the 28 days strength of concrete in plain water. A close examination of these curves indicates that the strength increases during the first 90 cycles of freeze-thaw in sea water as well as in plain water for all grades of concrete and after that it starts to decrease. The increase in strength upto the first 90 cycles may due to the fact that the specimens do not get saturated fully by sea water during this period. After 90 cycles, the specimens get saturated considerably by sea water and after crystallization of the salts together with their reaction with cementitious products within the body of concrete results in a significant decrease in the compressive strength. It is noted that in comparison to the 28 day compressive strength of plain water cured concrete at constant temperature of 27^{0}C, the compressive strength of the concrete specimens subjected to 30 cycles of freezing and thawing have been found to lie in the ranges of 87 to 92% for sea water and 93 to 98% for plain water; whereas the same value for the concrete specimens subjected to 180 cycles of freeze-thaw in sea water is 79 to 86% and in plain water is 90 to 96%.

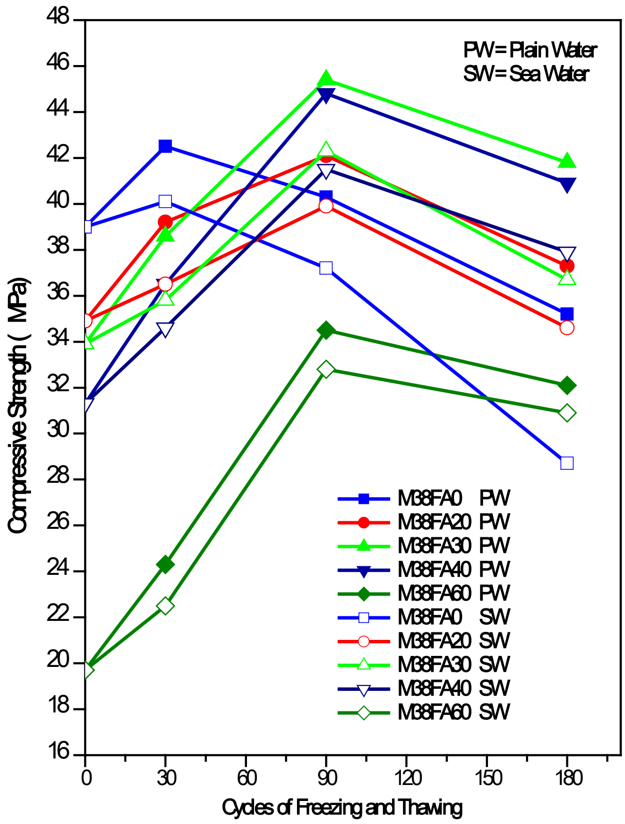

Fig.5: Compressive Strength - Freeze Thaw Relation for M38 Grade Concrete

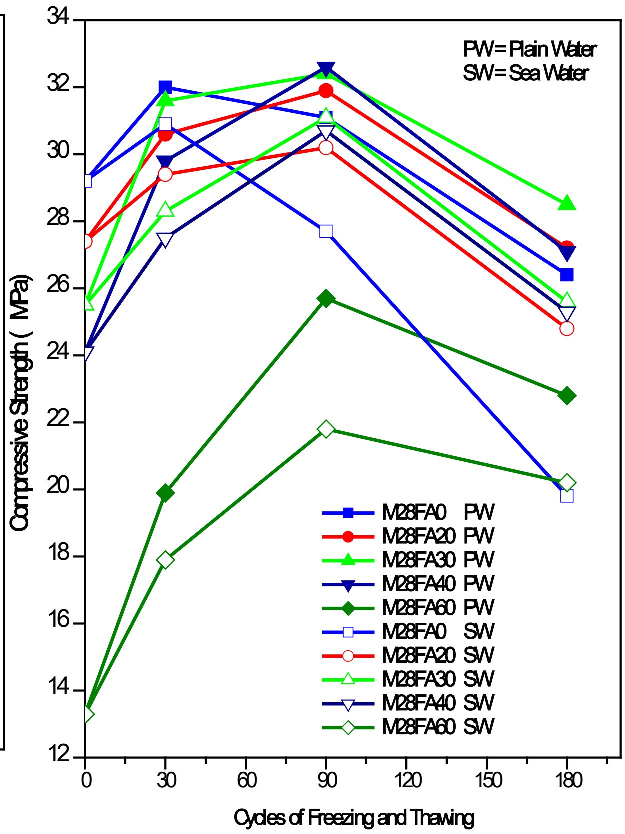

Fig.6: Compressive Strength - Freeze Thaw Relation for M28 Grade Concrete

At early ages of curing, OPC concretes i.e. no fly ash concrete achieves relatively higher compressive strength as compared to fly ash concrete. Test result shows that after 30 cycles of freeze-thaw, compressive strength for OPC concrete is around 8%, 9%, 14% and 43% higher than M38FA20, M38FA30, M38FA40 and M38FA60 concrete respectively. At initial age of curing, compressive strength is seen to decrease with the increase of fly ash content when compared with OPC concrete. For relatively larger freeze thaw cycles, compressive strength of the fly ash concrete specimens up to 40% cement replacement level are higher than that of OPC concrete. Compressive strength after 180 cycles of freeze-thaw for M38FA20, M38FA30 and M38FA40 concrete is higher by around by 6%, 19% and 16% respectively than OPC concrete. Cement normally gains its maximum strength within 28 days. During that period, lime produced form cement hydration remains within the hydration product. Generally, this lime reacts with fly ash and imparts more strength and for this reason, concrete made with fly ash will have lower strength than cement concrete at early ages of curing and higher strength at the later ages of curing. Conversely in cement concrete, this lime would remain intact and with time it would be susceptible to the effects of weathering, loss of strength and durability. M28 grade concrete also shows almost similar trend. After 180 cycles of freeze-thaw in plain water, compressive strength for M38FA20, M38FA30 and M38FA40 concrete are respectively 6%, 19% and 16% higher than the corresponding OPC concrete; whereas the same strength for M28FA20, M28FA30 and M28FA40 concrete are respectively 3%, 8% and 4% higher as compared to OPC concrete of similar grade.

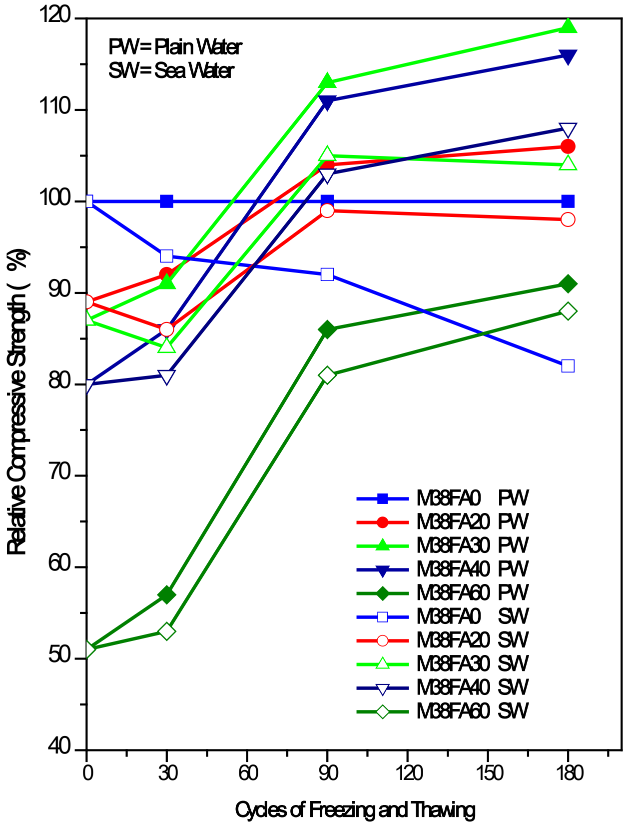

Fig.7: Relative Compressive Strength - Freeze Thaw Relation for M38 Grade Concrete

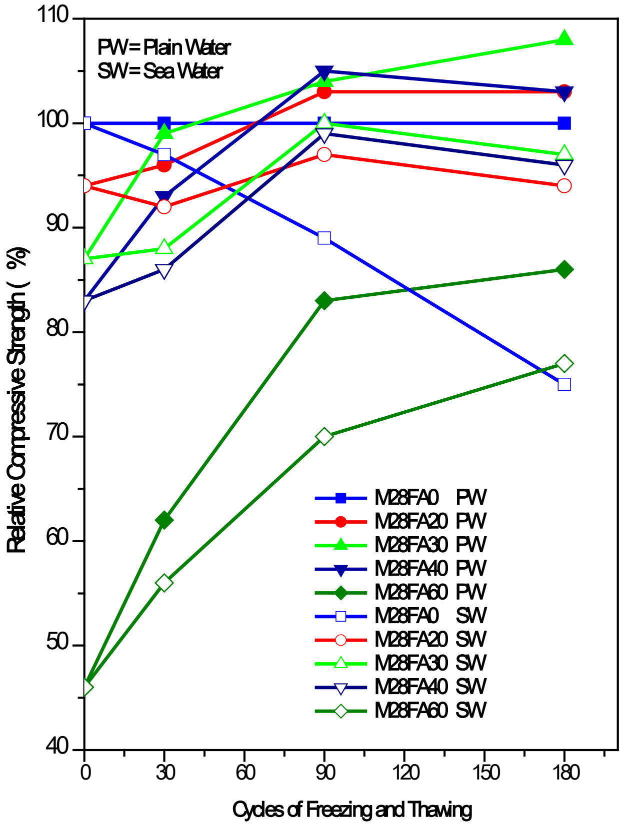

Fig.8: Relative Compressive Strength - Freeze Thaw Relation for M28 Grade Concrete

Test results also show that compressive strength of both OPC and fly ash concrete is reduced when it is exposed to seawater as compared to plain water curing. At the end of 90 cycles of freeze-thaw, compressive strength for M38FA20, M38FA30 and M38FA40 concrete is around 8%, 16% and 15% higher than 28 days compressive strength of OPC concrete when cured in plain water; whereas the same value is around 2%, 8%, and 6% higher for M38FA20, M38FA30 and M38FA40 concrete respectively as compared to OPC concrete when cured in sea water. Effect of seawater on the compressive strength of fly ash concrete has also been explained in terms of relative strength. In case of 180 cycles of freeze-thaw, reduction of strength is 10%, 7%, 2%, 6% and 22% for M28FA0, M28FA20, M28FA30, M28FA40 and M28FA60 concrete exposed to plain water; whereas the same value is 32%, 15%, 12%, 13% and 31% for the same concrete exposed to seawater respectively as compared to 28 days strength of OPC concrete in plain water. Fly ash concrete shows better resistance against strength deterioration compared to OPC concrete for larger freeze-thaw cycle.

Rate of strength deterioration for different types of concrete is observed to vary with the grade of concrete and is lower for the higher grade concrete. Among all the concrete studied, after 180 cycles of freeze-thaw in seawater deterioration of compressive strength deterioration is about 26%, 11%, 6%, 3% and 21% for M38FA0, M38FA20, M38FA30, M38FA40 and M38FA60 concrete respectively as compared to 28 day strength of OPC concrete; whereas the same strength is seen to decreased by around 32%, 15%, 12%, 13% and 31% for M28FA0, M28FA20, M28FA30, M28FA40 and M28FA60 concrete respectively compared to the 28 days strength of no fly ash concrete. Overall observation shows that, strength gaining for M38 grade concrete is around 5% and 7% higher as compared to M28 grade concrete in plain water and sea water respectively, which indicates that compressive strength gaining is relatively faster for higher grade concrete as compared to lower grade concrete.

5. CONCLUSIONS

Based on the limited number of tests, variables studied over 180 freeze-thaw cycles and the subsequent discussions, the following conclusions are drawn:

(1) Concrete exposed to freeze-thaw cyclic loading in sea water is much more vulnerable to deterioration including erosion, splitting and crumbling than in plain water.

(2) The loss in weight of concrete specimens is found to the extent upto 6.6% in sea water and around 0.6% for the specimens placed in plain water due to substantial erosion and splitting of specimens in the freeze-thaw environment. Fly ash concrete shows better resistance against weight change as compared to OPC concrete.

(3) Fly ash concrete has better resistance regarding dimensional instability as compared to OPC concrete. Concrete specimens show significant decrease in volume as much as 0.83% in sea water under freeze-thaw environment; whereas the similar specimens exposed to plain water show a decrease in volume of around 0.21%.

(4) Compressive strength of concrete is seen to be affected the most under the freeze-thaw action. Sea water causes the most detrimental effect on the compressive strength of concrete, the loss being of the order of 7% in plain water and 13% in sea water after 180 cycles of freeze-thaw as compared to the strength of normal plain water cured concrete. The study reveals that 30% and 40% blending of fly ash exhibited the best results with respect to resistance against compressive strength deterioration.

(5) Higher grade concrete showed better resistance against strength deterioration as compared to lower grades of concrete.

6. REFERENCES

Achintya and Prasad M. (2003). “Behavior of concrete in freeze--thaw environment of sea water.” J. Inst. Eng. India Civil Eng. Div., 96-101.

ASTM C666. Standard test method for resistance of concrete to rapid freezing and thawing. Annual Book of American Society for Testing Materials Standards, West Conshohocken, PA.

BS EN 12390-3. (2009). Testing of hardened concrete, Part 3: Compressive strength of test specimen, 1-19.

Dhir, R. K., Jones, M. R. (1999). “Development of chloride–resisting concrete using fly ash.” Fuel. Vol. 78(2), 137-142.

Diao. B., Sun. Y., Cheng. S. H., Ye. Y. H. (2011). “Effect of mixed-erosion freeze-thaw cycles and persistent loading on the behavior of reinforcement concrete beams.” Journal of Cold Reg. Eng. Vol. 25(1), 37–52.

Djerbi. A., Bonnet. S., Khelidj. A., Baroghel-bouny. V. (2008). “Influence of traversing crack on chloride diffusion into concrete.” Journal of Cement Concrete Research, Vol. 38(6), 877–883.

Drahushak-Crow, Roselle, and Von Fay Kurt F. (1991). “Freeze-thaw durability of concretes with various fly ashes.” Proceedings of the workshop on Coal Ash Utilization, Milwaukee, Sponsored by CBU, EPRI, and CANMET, April 22-23.

Hale, M., Freyne, S. and Russell, B. (2009). “Examining the frost resistance of high performance concrete.” Journal of Construction and Building Materials, Vol. 23(2), 878-888.

Litvan, G. G. (1976). “Frost action in cement in the presence of deicers.” Journal of Cement Concrete Research, Vol. 6(3), 351-356.

Mayers. J. J., Holm. G. H., and Mc. Allister. R. F. (1969). “The Hand Book of Ocean and Underwater Engineering.” McGraw-Hill London, 1094.

Neville, A. M. (2003). “Properties of Concrete.” Pearson Education Limited. Edinburgh. 844.

Penttala. V. (2006). “Surface and internal deterioration of concrete due to saline and non-saline freeze-thaw loads.” Journal of Cement Concrete Research, Vol. 36(5), 921-928.

Setzer, M. J. (1976). “Action of frost and deicing chemicals: Basic phenomena and testing.” Freeze--thaw Durability of Concrete, J Marchand. M Pigeon. and M Zetzer. eds. E. & F. N. Spon. London, Vol. 3, 21.

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EFFECTIVENESS OF COLOR COATING ON CORROSION OF STEEL REINFORCEMENT IN CONCRETE

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ABSTRACT

Corrosion of reinforced concrete structures is an increasing concern because it requires immediate repairs and rehabilitation to extend the service life of structure. So, there is increasing interest to use of different type of coating to reduce the intensity of corrosion. This study focused on effectiveness of color coating on corrosion of steel reinforcement in concrete. Corrosion induced crack initiation and propagation of crack, degree of corrosion and penetration rate are investigated experimentally in this paper. In this research work, red oxide, synthetic enamel paint and aluminum paint were used as a color coating. A total twenty four reinforced concrete specimens were constructed. These specimens were reinforced by one of No.4reinforcing bar. Electrochemical corrosion test was performed at room temperature as an accelerated corrosion test. To depasify the steel reinforcement, 3% NaCl solution was exploited as an electrolyte. The results showed that current density ($\mu A/cm^2$) for aluminum paint induced specimen demonstrate 63% lower result than non-coated specimen and red oxide and synthetic enamel paint show also 37% and 52% better result than non coated specimens. Aluminum paint increases the time of crack initiation 64% than non-coated specimens and others also show good results. Measured mass loss for 30.18 MPa and 36.23MPa strength induced concrete show average 67.5% and 56.25% lower result than predicted mass loss (according to the Faraday's law). It is also evident that aluminum paint induce specimens consume average 61% more time and provide 52.23% better penetration rate (mm/year) than non-coated specimens. However, these three types of color coating play an important role to protect the steel of reinforced concrete structure from severe environment.

Keyword: Corrosion, color coating, electrochemical corrosion test, current density, propagation of crack.

1. INTRODUCTION

Corrosion of embedded steel reinforcement is the most predominant factor of deterioration of reinforced concrete structures. Structure is manifested by the loss of structural serviceability that is characterized by concrete cracking and delamination. Corrosion of steel reinforcements generate the concrete cracking and eventual spalling of cover concrete, reduction in cross section of steel reinforcements (Mehta and Monteiro 1997). So the study of corrosion is necessary to assess the durability of structure over time (Mullard and Stewart 2011). It is also necessary to take the consideration both serviceability and strength (Val et al. 2009). Amount of corrosion products penetrates into the concrete pores before crack initiation. Hairline cracks of width less than 0.05 mm are found out immediately after cracks initiation. These products do not fully fill corrosion induced cracks in concrete immediately after their initiation. The cracks are filled by corrosion product gradually over time. The time required for the formation of cracks of 0.3 mm width is about ten times longer than the time between corrosion initiation and hairline cracking (Stewart 2001). An expansive pressure is attributed by corrosion products. This pressure creates the longitudinal cracking and spalling of concrete and debonding the reinforcement bars from concrete (Li and Yang 2011).

BS 8110 states that for appearance and corrosion, the maximum surface width of crack should not exceed 0.3 mm (British Standards Institution (BSI) 1985).ACI 318-09 states that limit crack widths to 0.40 mm, although it is recognized that corrosion is not correlated with surface crack widths (ACI 2009). Previous research on corrosion has mainly been concerned with the causes and maintenance of reinforcement, as well as to repair techniques and materials. A research program is conducted by both corrosion cracking and effectiveness of color coating on corrosion. Specimens with and without color coating were constructed. The study obtained corrosion by impressed current techniques (Electrochemical corrosion) as an accelerated corrosion techniques. This method is used so that corrosion test can be accomplished within a reasonable time. Degree of corrosion that is considered as one of the main parameter to predict durability of structure can be measured by the electrochemical corrosion test. Electrochemical corrosion test is done directly as a measurement of the electric current generated by the anodic reaction:

$$Fe \longrightarrow Fe^{2+} + 2e^-$$

and consumed by the cathodic reaction:

$$H_2O + 1/2O_2 + 2e^- \longrightarrow 2OH^-$$

and then converting the current flow by Faraday's law to mass loss of metal bar: $\Delta w = MIt/nF$, where Δw= mass loss of steel bar (g); I= applied current (A); t=time (s); n= ionic charge; M= atomic weight of steel (g); and F= Faraday's constant (96500 Columb/second).

From the above studies, it is understood that corrosion of steel reinforcement generates a major problem for structures. Considerable research has been carried out on the influence of cracking on corrosion, corrosion mechanism and techniques, influence of epoxy coating, organic coating and epoxy reign as protection material on steel corrosion. An attempt to understand the effect color coating on crack initiation of concrete, propagation rate of concrete crack, mass loss and penetration rate of steel reinforcement due to corrosion are the objectives of this paper.

2. EXPERIMENTAL PROCEDURE

This section describes the experimental program that was used to evaluate the effectiveness of color coating on corrosion of steel in reinforced concrete. Table-1 summarizes the experimental program. A total of twenty four cylindrical reinforced concrete specimens were constructed. The test parameters were the non-coated surface of steel bar and three different colors coating (Red Oxide, Synthetic Enamel Paint and Aluminum paint) and coarse aggregate (crushed burn brick and crushed stone). These specimens were divided into eight groups (A, B, C, D, E, F, G, and H), each with three identical dimensioned specimens. The specimens of each group were subjected to the same parameter. The specimen fabrication, material properties and test setup are presented in this section.

Table-1: Test Matrix

Group	Dimension (dia. x height) (mm x mm)	Diameter of Steel (mm)	Coating	Coarse aggregate	Time for induced corrrosion (hour)
[A]	100 x200	12	No coating	Crushed burn brick	258
[B]	100 x200	12	Red oxide	Crushed burn brick	330
[C]	100 x200	12	Synthetic enamel paint	Crushed burn brick	570
[D]	100 x200	12	Aluminum paint	Crushed burn brick	666
[E]	100 x200	12	No coating	Crushed stone	282
[F]	100 x200	12	Red oxide	Crushed stone	594
[G]	100 x200	12	Synthetic enamel paint	Crushed stone	642
[H]	100 x200	12	Aluminum paint	Crushed stone	738

2.1 Test specimen Fabrication

The detail of test specimen is shown in Fig.1. Each cylindrical specimen was 200 mm long with a 100 diameter. Each specimen was single reinforced with one No. 4 (12 mm diameter) deformed steel bar that was placed on the centre of cylindrical concrete. A total length of steel bar was 325 mm and a length of 150 mm of this bar was embedded into the cylindrical concrete. A 50 mm clear spacing was provided from the bottom of this specimen. Before four days of construction of the specimens, all steel bars were prepared according to the color coating. Construction of the specimens was undertaken in several stages. In the first stage, the steel cylindrical molds were oiled and steel bars were placed at the center of the mold. The steel bar extended 175 mm from the end of the cylindrical concrete to allow for electrical connection to impress the current. All specimens were cast vertically, compacted by a No. 5 (16 mm diameter) plain steel bar and trowel finished. After 24 hour of casting concrete specimens were demolded and cured these specimens at fresh water bath for 28 days and room temperature was $30^0 \pm 2^0$C.

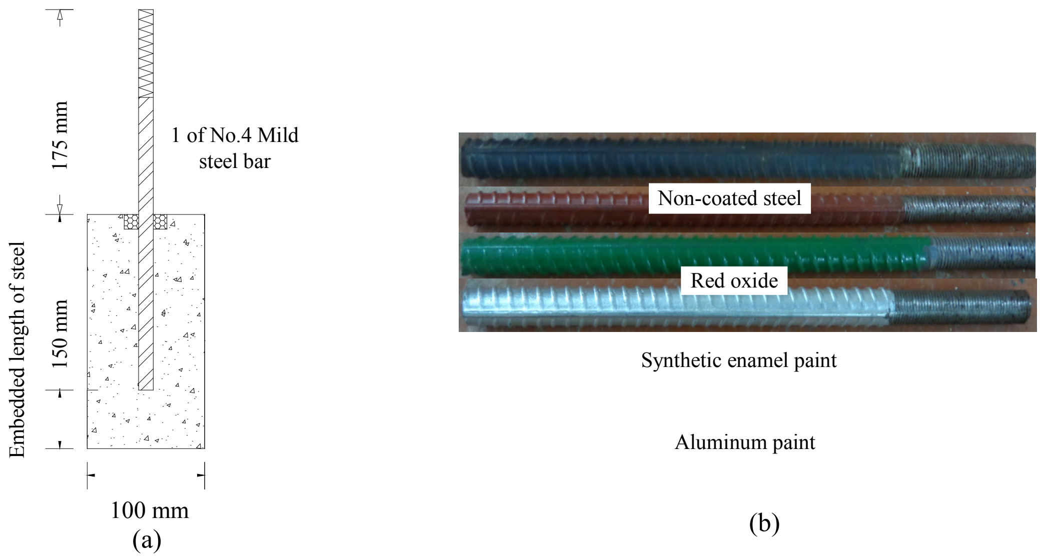


Figure 1: (a) Details of test specimen, (b) general view of color coating on steel

2.2 Material Properties

Normal strength concrete was used in this study with mix proportion by weight as follows (Cement: fine aggregate: coarse aggregate; 1:1.4:2.5) and water-cement ratio was 0.5. A 28 days average compressive strength of group A, B, C, and D specimens was 30.18 MPa and average compressive strength of group E, F, G, and H was 36.23 MPa. Splitting tensile strength of group A, B, C, and D specimens was 3.71 MPa and splitting tensile strength of group E, F, G, and H was 4.53 MPa.

Table 2: Mix Proportion of Concrete

Aggregate type	Water to cement ratio (%)	Cement (kg/m^3)	Water (kg/m^3)	Fine aggregate (kg/m^3)	Coarse aggregate (kg/m^3)	Compressive strength (MPa)
Crushed burn brick	50	295	147	435	516	30.18
Crushed stone	50	295	147	435	763	36.23

2.2.1 Aggregate Materials

Crushed burn brick with a maximum size of 19 mm (0.75 in) were used a coarse aggregate for group A, B, C, and D. The specific gravity was 2.04, and absorption capacity were 8.2%. The dry rodded unit weight was 1013 kg/m^3. Crushed stone with a maximum size of 19 mm (0.75 in) was used as a coarse aggregate for group E, F, G, and H. The specific gravity was 2.34, and absorption capacity was 1.2 %. The dry rodded unit weight was 1495 kg/m^3. Graded river bed sand was used for all groups as a fine aggregate with fineness modulus 2.72 in accordance with ASTM C136. The specific gravity was 2.64 and absorption capacity was 3.10%. The dry rodded unit weight was 1520 kg/m^3.

2.2.2 Steel materials

Steel reinforcement was Grade 72.5 deformed bar with diameter 12 mm (0.5 in). Average yield stress of the steel bar was 530 MPa and average ultimate stress was 622 MPa.

2.2.3 Steel Coating

Three different types of color coating were used in this study. These were Red oxide, Synthetic enamel paint and Aluminum paint. The specimens of group A and E were without any coating. Application of different color coating is shown in Table-1. The coating was polished on the clean deformed steel surface before 4 days of specimen construction. Fig. 1(b) shows the general view of color coating of steel.

2.2.4 Cement Materials

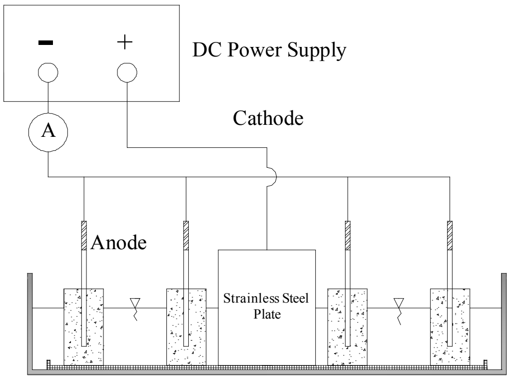
Portland composite cement was used for all specimens. Ingredients of Portland composite cements were clinker (65%-79%), slag, fly ash and limestone (21%-35%) and gypsum (0%-05%). Table 3. explicates the oxide compounds of Portland composite cement.

Table 3: Chemical composition of Portland composite cement

Compound	Na_2O	MgO	Al_2O_3	SiO_2	P_2O_5	SO_3	K_2O	CaO	TiO_2	Fe_2O_3
Conc. Unit	0.170%	1.860%	5.810%	21.03%	0.125%	1.7%	0.470%	60.89%	0.207%	3.079%

2.3 Test Setup and Procedure

To simulate corrosion in the laboratory, an impressed current technology was applied using variable power supply to maintain a constant voltage. A 3% salt was mixed with water to prepare electrochemical electrolyte. A group of reinforced concrete specimens were set on a wire meshes which placed at the bottom of this electrolyte cell. The specimens were placed as an anode and a stainless steel plate and wire mesh acted as a cathode. The current flowed through the steel bars across the concrete cover to a cathode (wire mesh and stainless steel plate). Circuit diagram is shown in Fig.2(a). After the application of applied current, current was measured regularly by digital multimeters for each specimen. After few days when initial crack was initiated on the surface of concrete body, first crack was measured. With initiating crack, cracks were measured regularly after 24 hours later. Specimen was removed from the corrosion cell when exhibited the target crack width. That time a false pullout specimen was placed in this place as a replacement specimen. Due to this cause applied current was behaved as previous. The steel reinforcement bars were pre-weighted before testing. After retrieving the specimens from the corrosion cell, cleaned of rust from the steel surface and then weighed to determine the actual mass loss of the steel reinforcing bars. Penetration rate (mm/year) also determined from the mass loss of reinforcing bars.



(a)

(b)

Figure 2: (a) Typical layout of accelerated corrosion test, (b) general view of accelerated corrosion test

3. EXPERIMENTAL RESULTS & DISCUSSION

3.1 Influence of Color Coating on Current Density

As describe earlier, reinforced concrete specimens were subjected to accelerated corrosion by impressed voltage. Constant impressed voltage was supplied by a power supply. Current measurements were taken for all specimens after every 24 hours. Current was measured by digital multimeter. The specimens exhibited different current density ($\mu A/cm^2$). Fig 3 shows the average current density ($\mu A/cm^2$) versus specimen's type that means coated and non-coated specimens. Specimens of group A, B, C, D were made with crushed burn brick and average compressive strength of these groups was 30.18 MPa. Specimens of group E, F, G, H were made with crushed stone and average compressive strength of these groups was 36.23 MPa. It is obvious that higher current densities were measured as the compressive strength decreased. Because of higher compressive strength of concrete reduce the porosity and increase the resistivity of concrete. Current densities measured for group A, B, C, D were higher than group E, F, G, H. Fig. 3 also shows that current densities measured for the specimens subjected to color coating (Red oxide, synthetic enamel pain and Aluminum paint) were significantly lower than those measured for non-coated specimens (Group A and E).

3.2 Relation between Time of Crack Initiation and Elapsed Time of Corrosion

The crack due to the expansion of concrete was measured for every specimen. The first crack for each specimen was a hairline crack and that time was considered as a time of crack initiation. Crack was measured every 24 hour later. When the target crack was exhibited then the specimens were pulled out from the corrosion cell. That time was considered as an elapsed time.

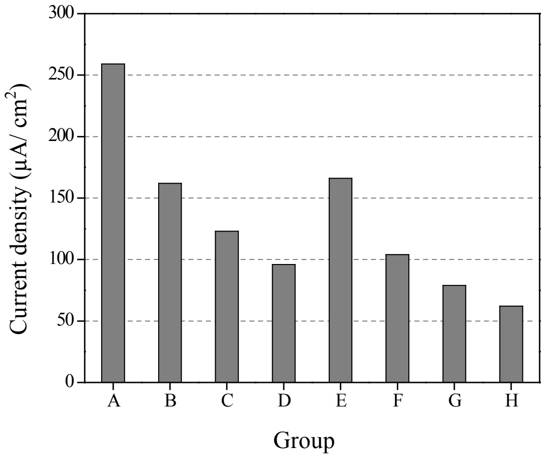


Figure 3: Current density (µA/ cm^2) versus specimen's relationship

Fig. 4 exemplifies the relationship between times of crack initiation and elapsed time of corrosion with the specimens. There have a 10% -13% difference in time of crack initiation between group A, B, C, D and group E, F, G, H. But color coatings have a significant effect on crack initiation of concrete. Fig. 4 shows that time of crack initiation for group A, B, C, D were 162, 186, 330 and 426 hours. It is cleared that group B, C, D takes 13%, 51% and 62% more time for crack initiation than group A. Time of crack initiation for group E, F, G, H were 186, 258, 402, 474 hours. And F, G, H takes 39%, 54% and 64% more time than group E. It shows that for different compressive strength of concrete, color coatings provide nearly similar results. These results conform that color coating provide the better resistance to protect the reinforced concrete from corrosion effect. Among the color coatings, aluminum paint presents better performance than others. This figure also shows the elapsed of corrosion for all group. The results of elapsed time clarify that color coatings are enlarged their elapsed time to get the nearly same crack width. Table 4 shows the final crack width of all group specimens.

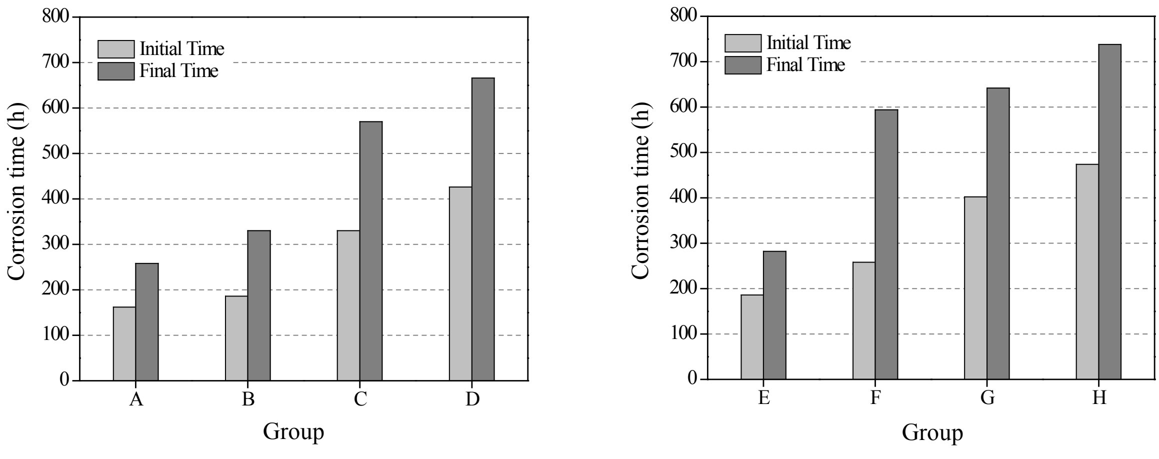


Figure 4: Corrosion time (h) versus specimen's relationship

3.3 Propagation of Corrosion Crack Width

The crack measurements were taken on the two sides (radial and longitudinal) of the specimens. The maximum crack width (mm) versus time relationship is illustrated in Fig. 5. In this figure, all groups crack data is stared from zero mm. After that measured cracks are plotted gradually 24 hours later. But times of crack initiation are different for all group illustrated in Fig. 4. Propagation of crack width (mm) measured for specimens subjected to color coatings were extensively lower than those measured for non-coated specimens like as group A and E. Fig. 6 shows that the percentage of propagation rate of crack width with different group specimens. In this figure, propagation rate of group A is considered as 100 percent. Propagation rate of other group specimens is calculated with respect to group A. This figure demonstrates that propagation rate of group E is 82.85 percent.

That means group E gives 17.15 percent better results than group A. The percentage of propagation rates of crack are 37.99, 29.08, 27.84 for group B, C, D and 30.58, 39.23, 24.14 for group F, G, H. These results explicate that group D and H award 72.16% and 75.86% better performance on propagation rate of crack width than group A and E respectively. Group D and H also show the better effect on propagation rate than group B, C and F, G respectively. But group B, C and group F, G also give good results than group A and E.

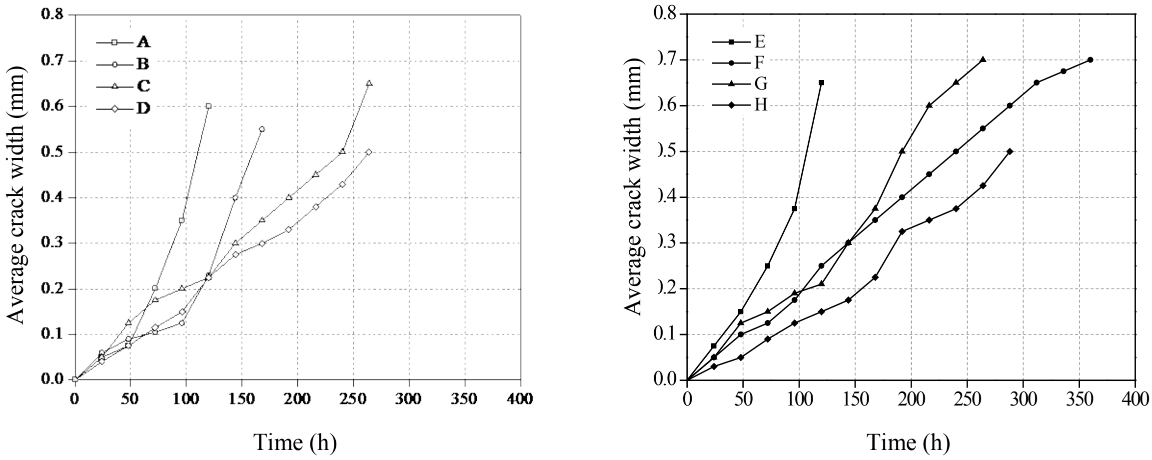


Figure 5: Average crack width (mm) versus time relationship

3.4 Relation between Measured Mass Loss and Predicted Mass Loss

According to the Faraday's law, predicted mass loss of corrosion was measured. Fig. 7 shows that the relationship between measured mass loss and predicted mass loss with respect to specimens. From this figure, it is initially seemed that there have no gradually increasing and decreasing relationships among A, B, C, D and also among E, F, G, H for measured mass loss. It is also shown the same nature for predicted mass loss. And results between crushed burn brick induced specimens (group A, B. C, D) and crushed stone induced specimens (group E, F, G, H) have no linear relationship.

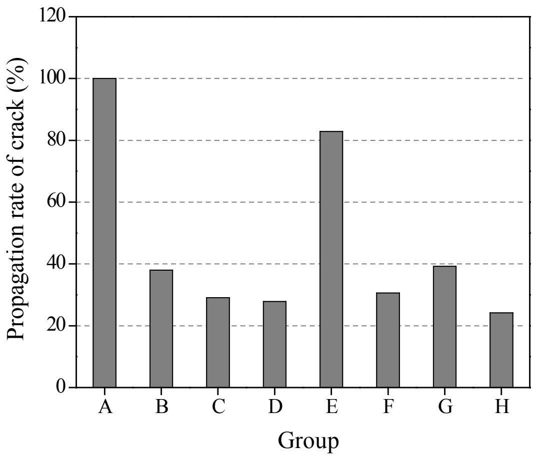


Figure 6: Percentage of propagation rate of crack width versus specimen relationship

Measured mass loss for group A, B, C, D are 72.17%, 68.45%, 65.2% and 65.29% lower than the predicted mass loss respectively. Measured mass loss for group E, F, G, H are 65.28%, 55.33%, 52.26%, 53.30% lower than the predicted mass loss. That means measured mass loss show lower result than predicted mass loss. Because of the reinforcing steel is protected by 44 mm high resistance concrete cover. It is cleared that difference of percentage of mass loss of group A, B, C, D show better performance on corrosion than group E, F, G, H. Predicted mass loss according to Faraday's law and measured mass loss are directly dependent on current density (µA/ cm2) and elapsed time of corrosion. Time of crack initiation and elapsed time of corrosion for group E, F, G, H were higher than group A, B, C, D respectively. So from the Fig. 7, it is initially seemed that the mass loss effect of group A, B, C, D presents better result but it is not authentic with respect to time effect.

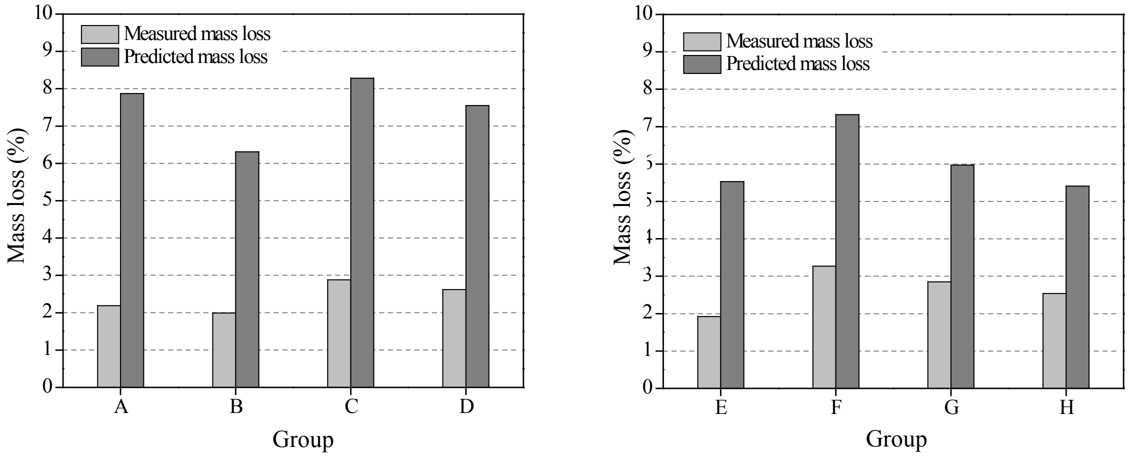


Figure 7: Relation between measured mass loss and predicted mass loss (Faraday's law)

3.5 Influence of Color Coating on Penetration Rate of Steel Reinforcement

The corrosion rate is calculated using the following equation given in ASTM G1 (Beaudoin et al. 2001):

$$\text{Corrosion rate (mm / year)} = \frac{(KxW)}{AxTxd}$$

Where K = a constant equal to 8.7 6×10^4, W =Mass loss in grams, A = actual corroded area of steel bar in cm^2 after removal from specimen and visually examining, T=time of exposure in hours, d=density of steel 7.85 (g/cm^3).

Table 4: Average maximum crack width and degree of corrosion (experimental and theoretical)

Group	Current density ($\mu A/cm^2$)	Time for induced corrosion (h)	Average Crack width (mm)	Propagation rate of crack (mm/hr)	Average % of mass loss		Penetration rate (mm/year)
					Measured	Predicted (Faraday's law)	
[A]	259	258	0.60	7.29×10^{-3}	2.19	7.87	234.26
[B]	162	330	0.45	2.77×10^{-3}	1.99	6.31	161.37
[C]	123	570	0.65	2.12×10^{-3}	2.88	8.28	138.71
[D]	96	666	0.50	2.03×10^{-3}	2.62	7.55	109.36
[E]	166	282	0.65	6.04×10^{-3}	1.92	5.53	196.12
[F]	104	594	0.70	2.23×10^{-3}	3.27	7.32	154.23
[G]	79	642	0.70	2.86×10^{-3}	2.85	5.97	123.16
[H]	62	738	0.50	1.76×10^{-3}	2.54	5.41	95.88

Fig. 8 shows that the penetration rate (mm/ year) versus elapsed time relationship. Penetration rate (mm/year) for group A, B, C, D were 234.26, 161.37, 138.71 and 109.36 respectively. Penetration rate (mm/year) for group E, F, G, H were 196.12, 154.23, 123.16 and 95.88 respectively. It is obvious that higher penetration rates (mm/year) were calculated for group A, B, C, D than group E, F, G, H. Penetration rate is also depended on the mass loss and time. But the results evident that group B, C, D give better perform than group A on penetration of steel reinforcement. And group F, G, H give also better result than group E. Elapsed time of group A was more little than others but others group show better result. Because of color coatings resist the applied current and longer the elapsed time. Among them group D and H provides 53.33% and 51.12% better performance than group A and E. That is the significant effect on corrosion protection of reinforced concrete structure.

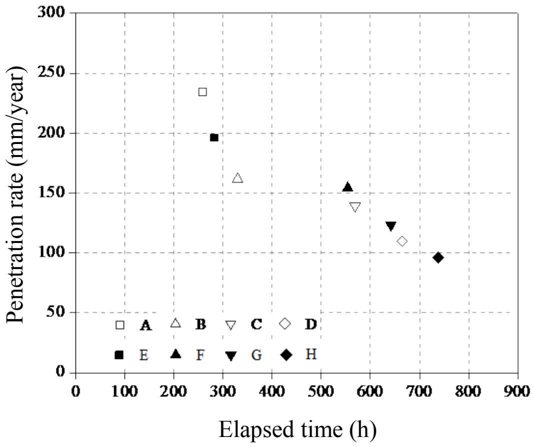


Figure 8: Penetration rate (mm/year) versus elapsed time relationship

4. CONCLUSIONS

The influence of different color coating on steel reinforcement of concrete was experimentally investigated. The study evaluated the concrete crack initiation, propagation of crack, mass loss and penetration rate due to corrosion. The main conclusions are as follows:

High strength concrete produces high electrical resistance to protect the corrosion of reinforcing steel of concrete. Current density ($\mu A/ cm^2$) for aluminum paint induced specimen show 63% lower result than non-coated specimen and red oxide and synthetic enamel paint show also 37% and 52% better result than non coated specimens. Crack initiation is the first sign of reinforcing steel corrosion. Color coatings play a significant role on crack initiation. Aluminum paint increases the time of crack initiation 64% than non-coated specimens and others also show good results. After 50 hours later of crack initiation, propagation of concrete crack of non-coated specimens propagates rapidly. But crack of aluminum paint induced specimens give a nearly linear relationship between crack width and time. Aluminum paint induced specimens provide average 74% better propagation rate than non-coated specimens. So, color coatings create a strong passive layer around steel surface that protect the concrete cover from corrosion induced crack initiation. Measured mass loss for 30.18 MPa and 36.23MPa strength induced concrete show average 67.5% and 56.25% lower result than predicted mass loss (according to the Faraday's law). But there have no linear relationship between mass loss and specimens. Because of mass loss depend on corrosion time and current density. Plot of penetration rate (mm/year) versus time gives a good relationship among the specimens. It is evident that aluminum paint consume average 61% more time and provide 52.23% better penetration rate (mm/year) than non-coated specimens. Red oxide and synthetic enamel paint also give better performance than non-coated specimens. These specimens consume.

REFERENCES

Mehta, P. K., and Monteiro, P. J. M. (1997). Concrete microstructure, properties and materials, 1st Ed., Indian Concrete Institute, Chennai, India

Mullard, J. A., and Stewart, M. G. (2011). "Corrosion-induced cover cracking: New test data and predictive models." *ACI Struct. J.,* 108(1), 71–79.

Val, D. V., Chernin, L., and Stewart, M. G. (2009). "Experimental and numerical investigation of corrosion induced cover cracking in reinforced concrete structures." *J. Struct. Eng.,* 135(4), 376–385.

Li, C.Q., and Yang, S.T. (2011). "Prediction of concrete crack width under combined reinforcement corrosion and applied load." *J. Eng. Mech.,* 137, 722-731.

Stewart, M. G. (2001). "Spalling risks, durability and life-cycle costs for RC buildings." *Proc., Int. Conf. on Safety, Risk and Reliability-Trends in Engineering. Malta*, IABSE, Zurich, Switzerland, 537-542.

British Standards Institution (BSI). (1985). "Structural use of concrete-Part 2: Code of practice for special circumstances." BS 8110-2, London.

American Concrete Institute (ACI). (2009). "Control of cracking in concrete structures." ACI 224.1R-07, Detroit.

Beaudoin, J.J., Ramachandran, V.S. (2001). Techniques for corrosion investigation in reinforced concrete. Noyes Publications, New Jersey.

PERFORMANCE EVALUATION OF RC BUILDINGS DESIGNED BY BNBC 2013

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ABSTRACT

Tectonically, Bangladesh lies at the junction of three tectonic plates - the Indian Plate, the Eurasian Plate and the Burma Plate. The Indian and Eurasian plate boundary has been the source of many past and present destructive earthquake events. Increasing frequency of earthquakes in recent times has created concern among experts and non-experts alike. BNBC (2013) is a major improvement from the previous code. Unlike the previous code, BNBC (2013) has provisions for both elastic and inelastic method of analysis of structures under strong ground motions. Equivalent Static Analysis, a simple but popular method of analysis, yields good result within elastic limit but cannot predict the behavior of the structure in inelastic region. Non Linear Static Analysis or Pushover Analysis, an approximate but better alternative, enables a better understanding of the performance of structures under earthquakes beyond elastic limit. In this study, the inelastic response of a structure designed by BNBC (2013) under seismic load has been investigated using pushover analysis. The scope of this study covers reinforced concrete buildings designed for four different seismic zones of Bangladesh under the action of three different levels of earthquake ground motion defined by ATC-40. The finding of the study confirms that reinforced concrete buildings designed as per BNBC (2013) meet all the performance criteria suggested by ATC-40. It has been also noted that structures designed for severe to very severe seismic intensity suffer more damage than structures designed for low to moderate seismic intensity.

Keywords: BNBC (2013), Nonlinear Static Analysis, Pushover Analysis, ATC-40, RC Buildings

1. INTRODUCTION

Earthquake is a dynamic event. During severe seismic events, structures are expected to deform inelastically. So to understand the behavior of structures subjected to strong ground motion caused by earthquakes, a dynamic analysis procedure which examines the structure beyond elastic limit should be employed. But methods satisfying both these criteria, such as nonlinear time history analysis, are time consuming and very complex for general design practices. On the other hand, popular tools for seismic design like equivalent static analysis cannot determine the post-elastic behavior of structures. Instead of using more complex and time consuming methods to evaluate the performance in inelastic region, Nonlinear Static Analysis gives a better insight of the performance of structures under earthquakes.

This work investigates the performance of bare frame concrete structures designed as per BNBC (2013) for four different seismic zones described in the code. The inelastic performances of the structures are evaluated according to the guidelines of ATC-40. The study has been carried out with the help of software ETABS Nonlinear v9.7.2.

2. METHODOLOGY

The study essentially consists of three parts: modeling and designing a concrete structure, perform a nonlinear static analysis and interpretation of results.

2.1 Modeling and Design

The structures are 15 story residential buildings located at four different seismic regions of Bangladesh with 25 feet bay in each direction in a 4x4 grid system. The height of the grade beam from the base is 6 feet. Typical story height is 10 feet. Height of the ground level is 12 feet. The slab thickness was calculated to be 7 inch. Steel ratios in columns are to be 1% to 3.4%. Building is designed as bare frame structure with strong column-weak beam condition. All supports are considered to be fixed support. Slabs are assumed not to carry any moment.

Building is assumed to be intermediate moment resisting frame. The structural members are proportioned to obtain code requirements and economy. BNBC (2013) is followed for defining loads. Self-weight of different members of the building has been calculated by the program itself. In addition to the self-weight, 80 psf loads are considered for partition walls and floor finish. On the beams at the perimeter, 5 inch thick brick wall has been assumed and assuming 10% opening, 0.45 k/ft line loads have been considered. To strictly concentrate on the earthquake resisting capacity of the structures, wind load is not considered in this study. Earthquake load is calculated as per BNBC (2013). To calculate seismic weight full dead load has been considered. Earthquake load is manually calculated and then added to the model as user defined load. The following factors and co-efficient are used:

Response Reduction Factor, R = 5 (For IMRF structure)
Seismic Zone Co-efficient, Z = 0.20 (Dhaka), 0.12 (Khulna), 0.28 (Chittagong), 0.36 (Sylhet)
Importance Factor, I = 1.0
Soil Type = SC
Site Depended Soil Factor, S = 1.15

2.2 Pushover Analysis

In pushover analysis, a building, which includes nonlinear properties, is subjected to monotonically increasing static loads under constant gravity loads. The choice of lateral load pattern is very important as the response is sensitive to it. The total procedure consists of constructing a response spectrum and a capacity spectrum.
Response Spectrum: An elastic response spectrum, for each earthquake hazard level of interest at a site based on seismic coefficients C_A and C_V is constructed. The seismic coefficient C_A represents the effective peak acceleration of the ground. A factor of about 2.5 times C_A represents the average value of peak response of a 5% damped short-period system in the acceleration domain. The seismic coefficient C_V represents 5% damped response of a 1-sec system and when divided by period defines acceleration response in the velocity domain. Then the response spectrum is converted in ADRS (Acceleration Displacement Response Spectrum) format with the equations provided in ATC-40. Finally, the response spectrum is adjusted for effective damping with the help of methods described in ATC-40 (ATC-40).

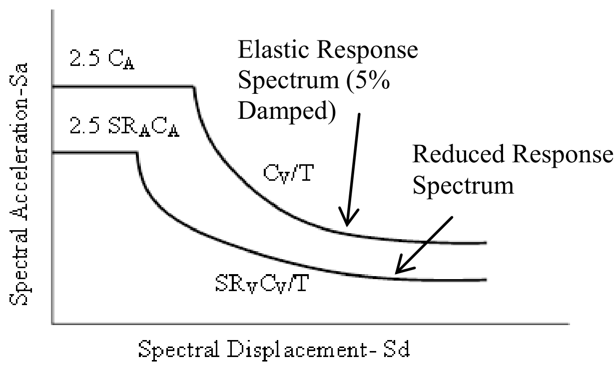

Figure 1: Response Spectrum

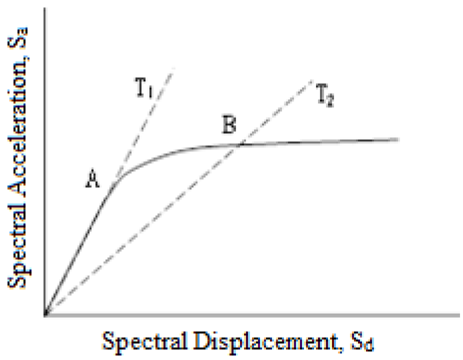

Figure 2: Capacity Spectrum

Capacity Spectrum: Nonlinear pushover analysis requires development of the capacity curve. This capacity curve is a plot of the total lateral seismic shear demand, "V", on the structure, at various increments of loading, against the lateral deflection of the building at the roof level, under that applied lateral force. The capacity curve is then converted into a capacity spectrum using relations from ATC-40. In this study the capacity spectrum has been directly achieved from ETABS.

2.3 Interpretation of Results

When the capacity spectrum and demand spectrum are superimposed, the intersection point is the performance point. A performance check verifies that structural and nonstructural components are not damaged beyond the acceptable limits of the performance objective for the force and displacement implied by the displacement demand. There are three procedures (A, B and C) to find performance point. In this study Procedure A has been used. It is the clearest, most transparent, and formula based analytical method, which can be easily programmed into a spreadsheet (ATC-40).
Performance of the structure is assessed against local (deformed shape and hinge formation) and global (displacement and lateral drift) criteria suggested in ATC-40 (1996).

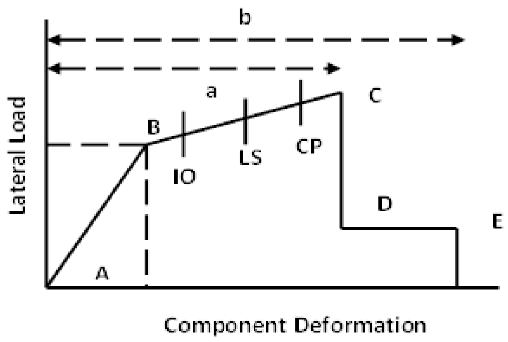

Figure 3: Force Deformation Action and Acceptance Criteria

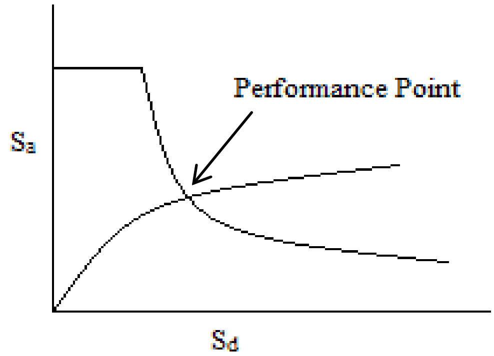

Figure 4: Performance Point

Each element is checked to determine whether its individual components satisfy acceptability requirements under performance point forces and deformations. Together with the global requirements, acceptability limits for individual components are the main criteria for assessing the building response. Figure 3 illustrates a generalized load-deformation relation appropriate for most concrete components. Point A corresponds to the unloaded condition. Point B represents yielding. Plastic deformation beyond point B is exhibited by the plastic hinges. Point C represents the ultimate capacity for pushover analysis. Point D represents a residual strength for pushover analysis. Point E represents total failure (ATC-40). Three local performance criteria, Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP), are located between point B and point C. ATC-40 defines three earthquake hazard levels: Serviceability Earthquake (SE), Design Earthquake (DE) and Maximum Earthquake (ME). It is expected that plastic hinge formation should not exceed IO, LS and CP for SE, DE and ME respectively.

It is also expected that story drift ratio should not be greater than 0.01 and 0.02 for serviceability earthquake and design earthquake respectively (ATC-40).

3. RESULTS AND DISCUSSION

From the models of four reinforced concrete buildings designed for four different seismic zones of Bangladesh, data of performance points, lateral drifts of a particular frame and nonlinear hinge were collected. These data have been show in graphical and tabular form for the ease of interpretation.

3.1 Performance Points:

Performance points for serviceability earthquake have been found within or very close to the elastic limits for all seismic zones. So, it can be safely concluded that structures, under the action of serviceability earthquake, remain elastic and no permanent damage takes place. To meet the demand of design earthquakes, all the buildings take some permanent damage, i.e., the demand spectra and capacity spectra meet slightly after the elastic limits. Under the action of maximum earthquakes, all structures deform well beyond the elastic limits and suffer significant permanent damages. But as performance point can be found, so structures do not undergo complete collapse.

It is also noted that structures designed for severe to very severe seismic zones yield more beyond the elastic regions of the capacity spectra to meet the demand spectra. So, structures designed for Sylhet or Chittagong suffers more damage than structures designed for Dhaka or Khulna.

3.2 Story Drift Ratio:

ATC-40 requires that story drift ratios for structures should not exceed Immediate Occupancy (IO) and Life Safety (LS) limits for serviceability and design earthquakes respectively. The corresponding story drift ratios are 0.01 and 0.02. Story drift ratio graphs from Figures 5, 6 and 7 show that these requirements are met. For Khulna, story drift ratios for design and serviceability earthquakes are essentially same. As the seismic intensity of different earthquake zones increases, story drifts also increases. But in all cases the recommended limits are not crossed.

3.3 Plastic Hinge Formation:

ATC-40 warrants that each component should satisfy the acceptability requirements under performance point forces and deformations. For serviceability earthquake, plastic hinge formation should be limited to IO-LS limit. For all four structures of this study, this criterion has been fulfilled. For design earthquake, it is expected that no

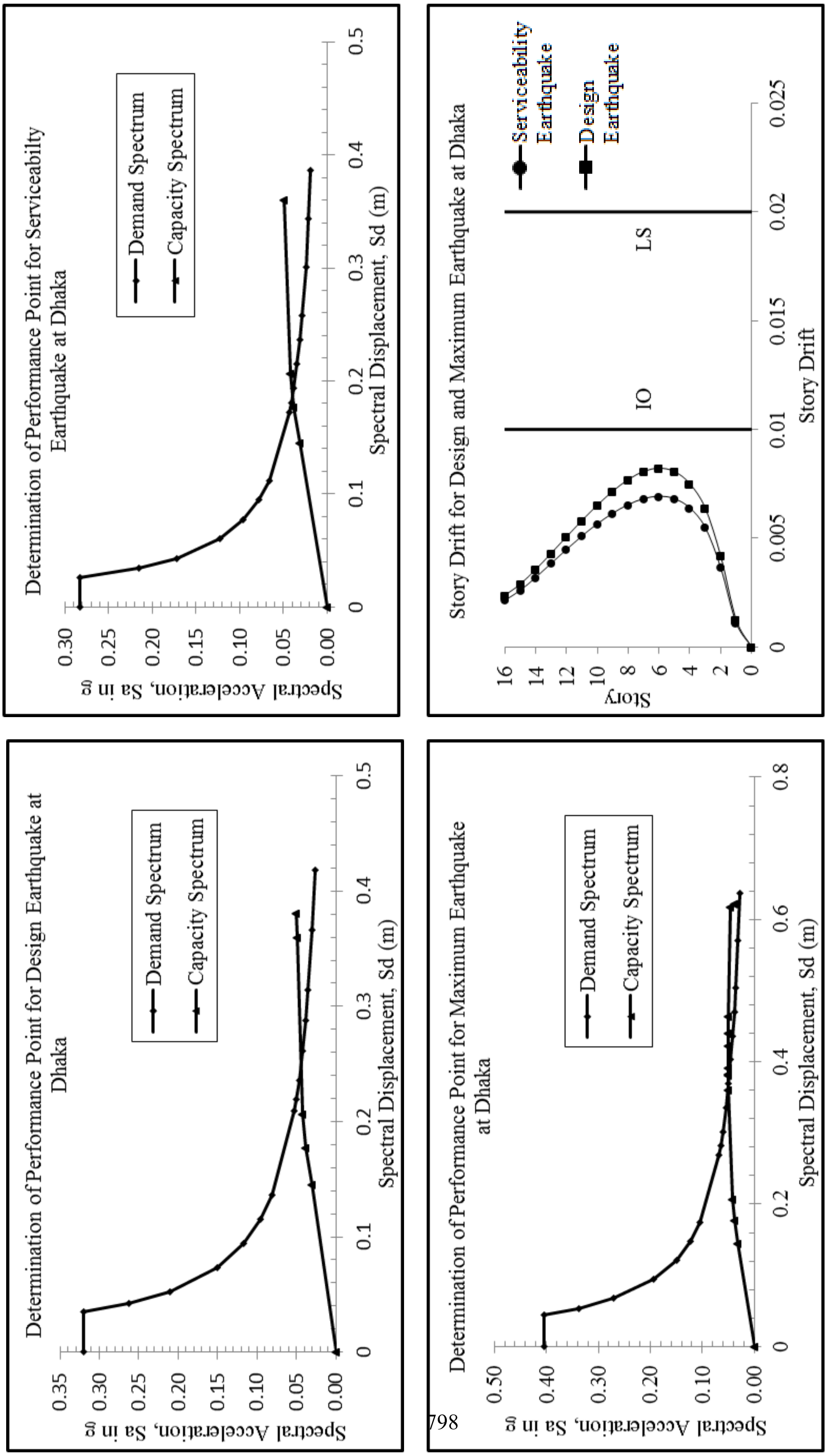

Figure 5: Performance Evaluation of a 15 Story Building Designed For Dhaka (Seismic Zone 2)

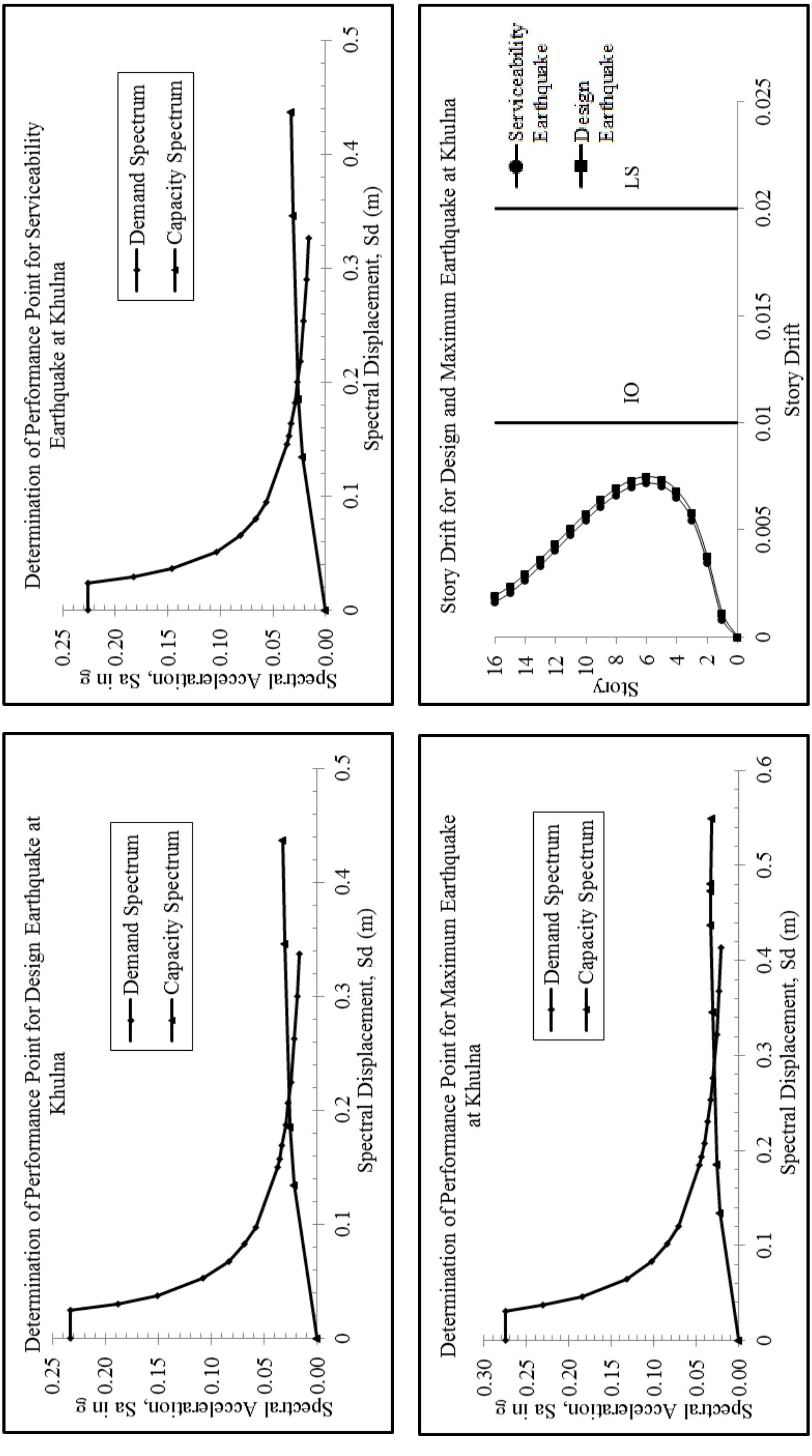

Figure 6: Performance Evaluation of a 15 Story Building Designed For Khulna (Seismic Zone 1)

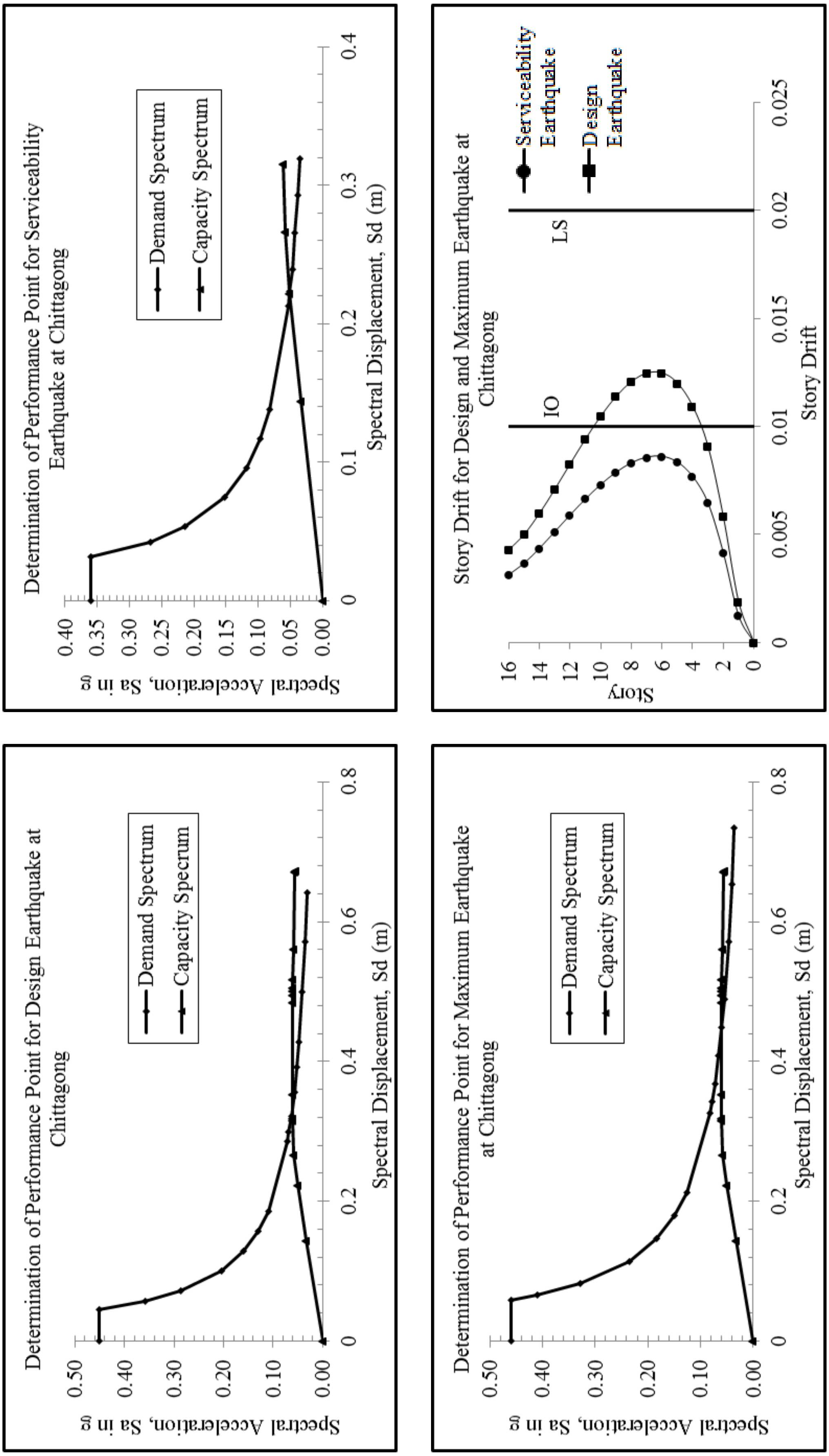

Figure 7: Performance Evaluation of a 15 Story Building Designed For Chittagong (Seismic Zone 3)

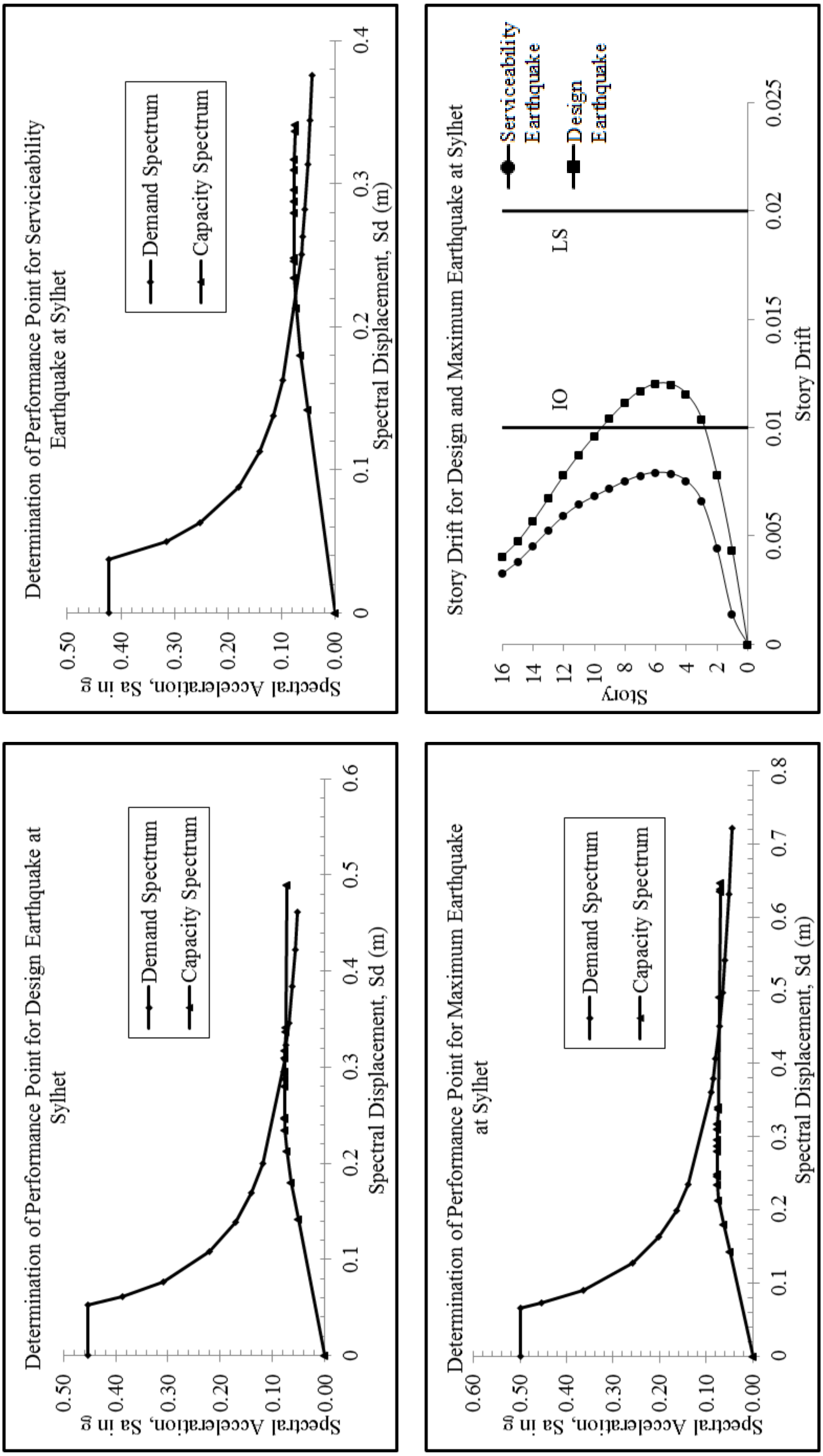

Figure 8: Performance Evaluation of a 15 Story Building Designed For Sylhet (Seismic Zone 4)

Table 8: Performance Points

Earthquake Ground Motion Level	Dhaka (Seismic Zone 2)		Khulna (Seismic Zone 1)		Chittagong (Seismic Zone 3)		Sylhet (Seismic Zone 4)	
	S_a (g)	S_d (m)	S_a (g)	S_d (m)	S_a (g)	S_d (m)	S_a (g)	S_d (m)
Serviceability Earthquake	0.039	0.183	0.028	0.200	0.051	0.232	0.074	0.223
Design Earthquake	0.042	0.255	0.029	0.204	0.059	0.328	0.076	0.317
Maximum Earthquake	0.051	0.350	0.030	0.289	0.061	0.484	0.072	0.490

Table 9: Plastic Hinge Formation Summery

Seismic Zone	Earthquake Ground Motion Level	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E	TOTAL
Dhaka (Seismic Zone 2)	Serviceability Earthquake	1873	270	0	0	0	0	0	0	2080
	Design Earthquake	1650	189	241	0	0	0	0	0	2080
	Maximum Earthquake	1618	210	234	18	0	0	0	0	2080
Khulna (Seismic Zone 1)	Serviceability Earthquake	1903	180	0	0	0	0	0	0	2080
	Design Earthquake	1899	196	0	0	0	0	0	0	2080
	Maximum Earthquake	1626	136	127	3	0	0	0	0	2080
Chittagong (Seismic Zone 3)	Serviceability Earthquake	1863	217	0	0	0	0	0	0	2080
	Design Earthquake	1630	410	40	0	0	0	0	0	2080
	Maximum Earthquake	1422	216	205	237	0	0	0	0	2080
Sylhet (Seismic Zone 4)	Serviceability Earthquake	1737	343	0	0	0	0	0	0	2080
	Design Earthquake	1569	470	41	0	0	0	0	0	2080
	Maximum Earthquake	1404	46	210	418	0	0	0	0	2080

plastic hinge should be formed beyond LS-CP range. All the structures fulfill this requirement also. No plastic hinge formed beyond the CP-C limit when structures suffered significant inelastic deformation to meet the seismic demand of maximum earthquake, as required.

4. CONCLUSION

For all the structures, performance points have been found within or just after the elastic limit for serviceability earthquake, well before the collapse but in the inelastic region for design earthquake and just before the collapse for maximum earthquake. Story drift ratios have been found within specified limit for all earthquake hazard levels. No plastic hinge has been found beyond the expected ranges. So it can be concluded that buildings designed as per BNBC (2013) for four different seismic zones of Bangladesh satisfies all the seismic performance requirements (local and global) prescribed by ATC-40 guidelines.

REFERENCES

ACI Committee, 318 (2008), Building Code Requirement for Structural Concrete, American Concrete Institute, Detroit, USA.

ATC (1996), Seismic Evaluation and Retrofit of Concrete Building (Volume I), Report (ATC-40), Applied Technology Council, California, USA.

BNBC (2013), Bangladesh National Building Code, House and Building Research Institute, Mirpur, Dhaka, Bangladesh.

Computer and Structures Inc. , ETABS Nonlinear Version 9.7.2, California, USA.

Das, S., (2012), Evaluation of Seismic Performance of Reinforced Concrete Structures by Pushover Analysis, Thesis Submitted to Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka, Bangladesh.

Sarfin, M. A. (2013), Performance Evaluation of RC Buildings Designed by BNBC 2013, Thesis Submitted to Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka, Bangladesh.

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COST OPTIMIZATION OF COLUMNS

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ABSTRACT

The ratio of longitudinal steel area to gross concrete section is in the range from 0.01 to 0.08, according to BNBC Code. The common practice is to choose an arbitrary section and check that for bending and axial load with a reinforcement ratio 2% but we don't know whether it is economical or not. However, for a particular moment and load there is only one section which is economical, it means only for a certain percentage of reinforcement the section is optimized. But this is not for a fixed percentage of reinforcement, because the section has its components i.e. concrete and reinforcement and the cost of these materials are different which may in-crease or decrease independently. The percentage of reinforcement in the optimized section varies with the price ratio of steel to concrete. Analysing for the present cost ratio of concrete and reinforcement, it is seen that a column section is optimized at 1 percent of reinforcement. This is true for every column that after a certain price ratio it is optimized at reinforcement of the order of 1 percent on that loading and moment condition. Use of high strength concrete in the column has an effect of minimizing the cost. Using 5000 psi concrete instead of 3000 psi concrete saves 20-50 percent of total cost in general. For same axial load and moment resisting capacity, a circular column is found to be more costly than a square column. Also, the cost differences between circular and square column increase with the increase in gross area of concrete.

Keywords: Optimization, Price ratios, High strength, circular, rectangular

1. INTRODUCTION

According to BNBC 1993, the ratio of longitudinal steel area (Ast) to gross concrete area (Ag) in column is in the range from 0.01 to 0.08. But, in practice, designers do not usually provide more steel than 4% of the gross concrete area (Ag) because of the difficulty owing to congestion of the reinforcement. The common practice in designing a column is to choose arbitrary sections and check them for bending and axial load with a rein-forcement about 1% to 4% of the gross concrete area. However, for a particular loading condition, there is on-ly one section which can be found economical, which means for a certain percentage of reinforcement the sec-tion is optimized. The percentage of reinforcement in the optimized section is found to vary with the price ra-tio of steel to concrete.

Use of high strength concrete in the column is another way to minimize the total cost of the column. High strength concrete reduces the requirement of high percentage of steel reinforcement in the column. Cost of steel being the governing factor in the optimization of the column section, the section gets optimized with the increase in strength of concrete. Therefore, the effect of percentage of reinforcement and the strength of con-crete both in the optimum design of column section is dealt with in this paper. Also, the cost comparison be-tween circular and rectangular column sections having same axial load and moment resisting capacity is ana-lyzed in this paper to select an economical section..

2. METHODOLOGY

Cost optimization analysis is carried out in this paper for three cases.

1. To find an optimized percentage of reinforcement considering existing price of concrete and reinforcement
2. To compare the cost of column for various strength of concrete
3. To compare the cost of circular and square column for similar loading condition.

For a specific loading condition, the required percentage of reinforcement will decrease with the increase in gross concrete area. This will decrease up to a certain section and after that the required amount of reinforcement will increase. To find the optimized section, columns of a particular building are chosen and designed for various section and percentage of reinforcement for a specific loading condition. Finally, total cost Vs percentage of reinforcement graph is plotted for different price ratios to find out the optimized percentage of reinforcement. ETABS 9.7 is used to design the column.

Again, in comparing the cost of column for different strength of concrete, column of a particular building is chosen and designed for different strength of concrete by ETABS 9.7. Finally, a graph of total cost VS strength of concrete is plotted to show the result.

For similar loading condition, both square and circular column are designed firstly by changing the concrete gross area and secondly by changing the percentage of reinforcement. The cost of ties in both cases is neglected as difference of cost of ties is very little. Finally, two graphs are plotted for either of the cases showing the variation of cost. PCA COL software is used here to design the column.

3. SELECTION OF STRUCTURE

The cost optimization analysis is performed for various types of buildings. The conclusions and remarks are given on the basis of the structures which have been dealt with. In this paper, the analysis results (tables and figures) are shown only for a six storied irregular type of building like figure 1. The column B4 of the building is analyzed here for optimum cost calculation.

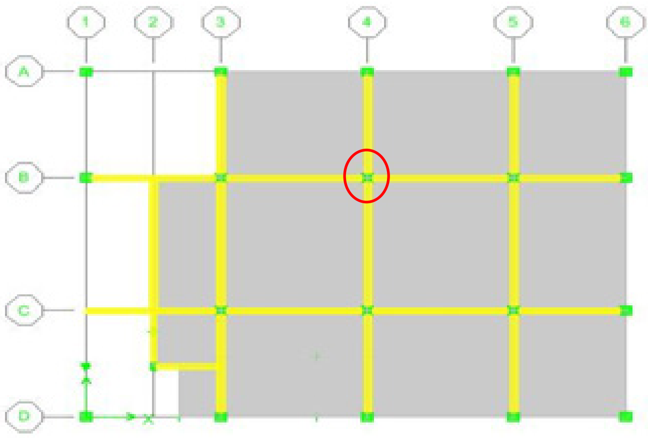

Figure1: Ground floor plan of a six storied irregular building from ETABS

4. COST ANALYSIS

For simplicity, a 10 ft column (Ground Floor) with 60 grade reinforcement and 4000 psi concrete is considered. The present cost of 60 grade steel and 4000 psi concrete are given below.

Cost of 60 grade Reinforcement = 60000 BDT/ton
Cost of 4000 psi concrete = 210 BDT/ cft

5. A OPTIMIZED PERCENTAGE OF REINFORCEMENT CONSIDERING EXISTING PRICE

The applied axial load and moment in the column resisted both by concrete and steel simultaneously. For a specific loading condition, if we increase the concrete gross section, the percentage of reinforcement will decrease. Since the price of steel controls the cost, the column section gets optimized with lower percentage of steel. As the cost of concrete and reinforcement may increase or decrease independently, the economic section is found to depend on a factor called price ratio X where,

X = Price of 1cft reinforcement (490 lb)/Price of 1 cft concrete

= Price of reinforcement (ton)/(Price of concrete (cft)×4.57) [1 ton steel =4.57 cft]

The following table shows the cost analysis of B4 column at ground floor of the selected building for different price ratios.

Table 1: cost analysis of B4 column for different price ratios

B	A_g	A_{st}	Percentage Of Steel	Total Cost (BDT) for X=5	Total Cost (BDT) for X=10	Total Cost (BDT) for X=15	Total Cost (BDT) for X=20	Total Cost (BDT) for X=25	Total Cost (BDT) for X=30	Total Cost (BDT) for X=50	Total Cost (BDT) for X=65
16	288	17.864	6.203	5503	6805	8108	9410	10713	12016	17226	21134
18	324	14.925	4.606	5813	6902	7990	9078	10166	11255	15608	18873
18	360	13.532	3.759	6237	7223	8210	9197	10184	11170	15117	18077
18	396	11.922	3.011	6644	7514	8383	9252	10122	10991	14468	17076
18	432	10.672	2.470	7078	7856	8635	9413	10191	10969	14082	16416
20	440	9.348	2.125	7098	7780	8462	9143	9825	10506	13233	15278
20	480	8.04	1.675	7586	8173	8759	9345	9931	10518	12863	14621
22	484	6.598	1.363	7539	8021	8502	8983	9464	9945	11869	13313
22	506	5.802	1.147	7802	8225	8648	9071	9494	9918	11610	12879
22	528	5.28	1	8085	8470	8855	9240	9625	10010	11550	12705

Two graphs are plotted from the above table to represent the result. Figure 1 implies for the comparison of cost for various price ratios and figure 2 shows the variation of optimizing percentage of reinforcement with the price ratios.

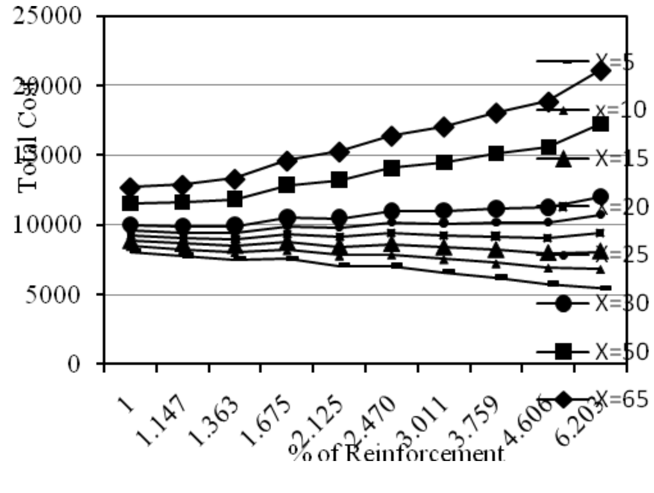

Figure2: Graphical analysis of cost for various price ratios

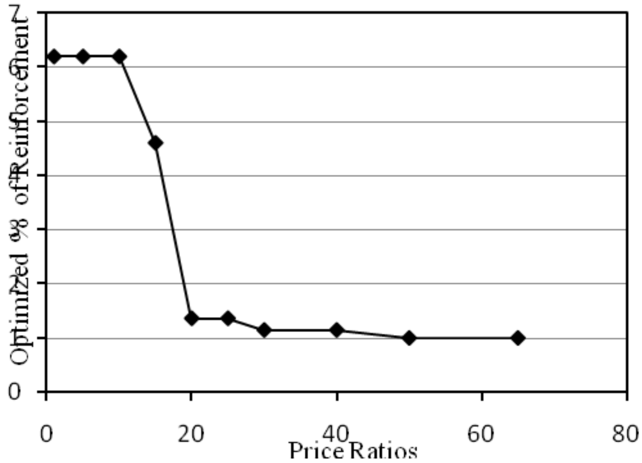

Figure 3: Variation of Optimized percentage of reinforcement with price ratios

For current price of reinforcement and concrete, the estimated price ratio is 65. It can be seen from the figure 1 that use of 1percent of reinforcement at the existing price ratio gives the most cost effective design of column and the estimated cost is found 12705 BDT. The corresponding cost for 2.47 percent of reinforcement is estimated 16416 BDT. The difference in cost is 3711 BDT may seem very small. But considering the overall structure, this amount will not be so small. For the B4 column of this particular building, use of 2.47 percent of reinforcement instead of 1 percent of reinforcement increases the column cost by 29.21percent. Again, column designed using 4.6 percent of reinforcement is about 1.5 times more costly than column designed using 1 percent of reinforcement.

Figure 2 shows how the optimized percentage of reinforcement changes with the changes of price ratios. For the same price of concrete and reinforcement per cft, the column section needs about 6.2 percent of reinforcement to get optimized. With the increase in price ratio, the optimized percentage of reinforcement decreases and after a certain price ratio the economical percentage of reinforcement becomes on the order of 1 percent. This is true for every column that after a certain price ratio it is optimized at reinforcement of the order of 1 percent on that loading and moment condition.

6. COST COMPARISON OF COLUMNS FOR VARIOUS STRENGTH OF CONCRETE

The same structure is now analyzed here for a given section of column at the ground floor level (10 ft) but for different price and strength of concrete. The analysis result for B4 column is summarized below in the table.

Table 2: Cost analysis of B4 column for different strength of concrete

Concrete Strength (psi)	Price of Concrete Per cft (BDT)	Column Size (in×in)	Reqd. Reinforcement (in^2)	Cost of Reinforcement (BDT)	Cost of Concrete (BDT)	Total Cost (BDT)
2000	195	20×20	22.397	20785	5417	26202
2500	198	20×20	19.177	17797	5500	23297
3000	201	20×20	15.876	14733	5583	20317
3500	205	20×20	13.101	12158	5694	17853
4000	210	20×20	10.547	9788	5833	15621
4500	215	20×20	8.059	7479	5972	13451
5000	220	20×20	5.849	5428	6111	11539
5500	223	20×20	4	3712	6194	9907

Price Source: Various Construction Firms, Bangladesh, August 2011

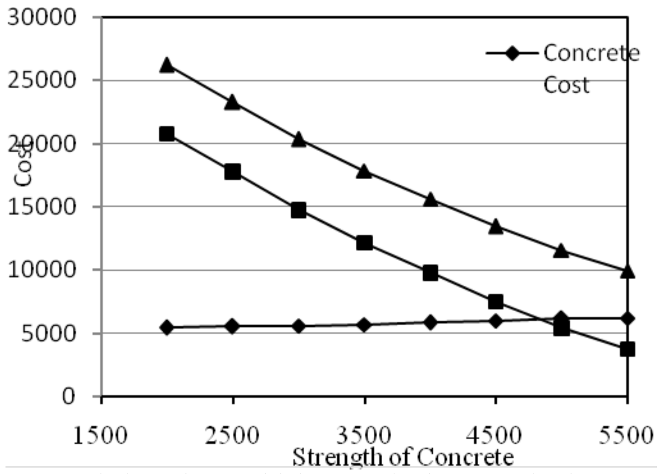

Figure 4: Variation of cost with increase in the strength of concrete

The analysis shows the required amount of reinforcement decreases with the increasing strength of concrete for a specific loading condition. The moment and axial load on a column is resisted by concrete and reinforcement simultaneously. As the selected column section is fixed and the loading condition is similar, so the required reinforcement decreases with the increasing strength of concrete. Form the graph, it is also seen that the decrease in cost of reinforcement is very small compared to the increase in cost of concrete. Also, the total cost reduces 2 times for this particular column when 5000 psi concrete is used instead of 3000 psi concrete. After performing the similar analysis for some high rise buildings, it is seen that using 5000 psi concrete instead of 3000 psi concrete saves 20-50 percent of total cost.

7. COMPARISON OF THE COST BETWEEN CIRCULAR AND SQUARE COLUMN FOR SIMILAR LOADING CONDITIONS

For architectural purposes, engineers frequently use circular column. Therefore, it is of prime importance to know the cost effectiveness of circular column with respect to square or rectangular column. For comparison purposes, 8 column samples are taken and are designed as both circular and square columns:

- Keeping the gross concrete area same but changing the required reinforcement
- Keeping the reinforcement same but changing the required gross area.

Table 3: Cost comparison between square and circular columns changing the gross concrete area

Sample	L	B	A_g (in^2)	No. of Bar	A_{st}	% of Steel	Axial Load Capacity	Moment Capacity M_x	Moment Capacity M_y	Cost of Concrete (BDT)	Cost of Reinforcement (BDT)	Total cost
A	12	12	144	8#6	3.52	2.44	128	73	73	2100	3267	5367
	Dia	16.75	220.5	8#6	3.52	1.6	128	74	74	3216	3267	6482
B	15	15	225	8#8	6.32	2.81	220	164	164	3281	5865	9146
	Dia	21.25	355	8#8	6.32	1.78	220	166	166	5177	5865	11042
C	16	16	256	8#8	6.32	2.47	252	190	190	3733	5865	9598
	Dia	22.75	406.5	8#8	6.32	1.55	252	194	194	5928	5865	11793
D	19	19	361	8#9	8	2.22	369	312	312	5265	7424	12689
	Dia	27	572.5	8#9	8	1.4	366	316	316	8349	7424	15773
E	20	20	400	8#9	8	2	413	350	350	5833	7424	13258

	Dia	28.35	638	8#9	8	2.01	413	359	359	9304	7424	16728
F	22	22	484	8#10	10.16	2.1	505	481	481	7058	9429	16487
	Dia	31.25	767	8#10	10.16	2.07	488	488	488	11185	9429	20614
G	24	24	576	8#10	10.16	1.76	611	585	585	8400	9429	17829
	Dia	34	908	8#10	10.16	1.77	602	595	595	13242	9429	22670
H	25	25	625	8#10	10.16	1.63	657	643	643	9115	9429	18543
	Dia	35.5	990	8#10	10.16	1.65	648	657	657	14438	9429	23866

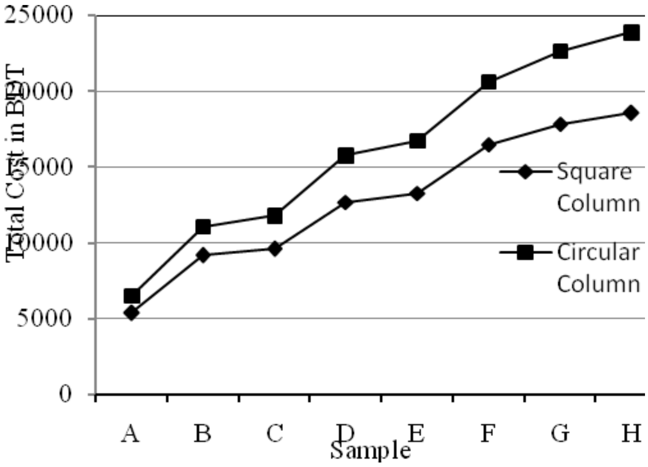

Figure 5: Difference in cost for various samples (Same reinforcement)

Table 4: Cost comparison between square and circular columns changing the reinforcement

Sample	L	B	A_g (in^2)	No. of Bar	A_{st} (in^2)	% of Steel	Axial Load Capacity (kip)	Moment Capacity M_x (k-ft)	Moment Capacity M_y (k-ft)	Cost of Concrete (BDT)	Cost of Reinforcement (BDT)	Total cost
A	12	12	144	8#6	3.52	2.44	128	73	73	2100	3267	5367
	Dia	13.5	143.1	8#10	10.16	1.6	128	74	74	2087	9429	11516
B	15	15	225	8#8	6.32	2.81	220	164	164	3281	5865	9146
	Dia	17	227	8#14	18	1.78	220	166	166	3310	16705	20015
C	16	16	256	8#8	6.32	2.47	252	190	190	3733	5865	9598
	Dia	18	254.5	11#11	17.16	1.55	252	194	194	3711	15925	19636
D	19	19	361	8#9	8	2.22	369	312	312	5265	7424	12689
	Dia	21.5	363.1	15#10	19.05	1.4	366	316	316	5295	17679	22974
E	20	20	400	8#9	8	2	413	350	350	5833	7424	13258
	Dia	22.5	397.6	16#10	20.32	2.01	413	359	359	5798	18858	24656
F	22	22	484	8#10	10.16	2.1	505	481	481	7058	9429	16487
	Dia	25	490.9	11#14	24.75	2.07	488	488	488	7159	22969	30127
G	24	24	576	8#10	10.16	1.76	611	585	585	8400	9429	17829

	Dia	27	572.6	12#14	27	1.77	602	595	595	8350	25057	33407
H	25	25	625	8#10	10.16	1.63	657	643	643	9115	9429	18543
	Dia	28	615.8	13#14	29.25	1.65	648	657	657	8980	27145	36125

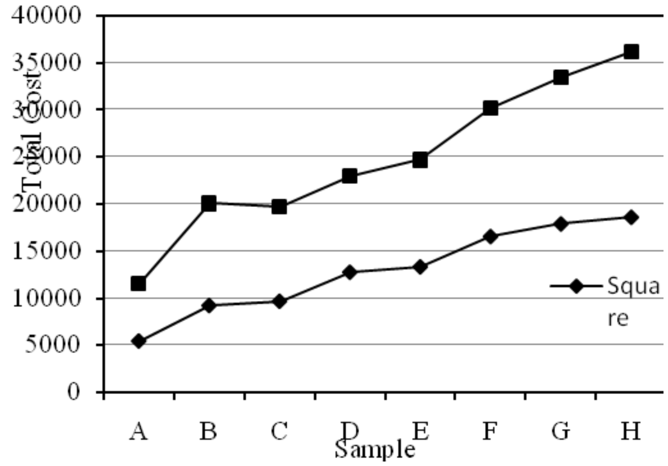

Figure 6: Difference in cost for various samples (Same concrete gross area)

Considering the same reinforcement (table 5), it can be seen that a circular column requires more concrete area than a square column for same loading conditions. Therefore, circular column costs about 20-30 percent more than square column in this case. Considering the similar gross concrete area (table 6), it can be seen that a circular column requires more reinforcement than a square column. Therefore, circular column costs about 80-130 percent more than square column in this case. Also, from the above two graphs it is clear that the cost differences between circular and square column increase with the increase in gross area of concrete.

8. CONCLUSION

When the price ratio is more than 30, column section gets optimized at reinforcement of the order of 1 per-cent. For present market price of concrete and reinforcement (X=65) in Bangladesh, a column section is optimized at 1 percent of reinforcement. This implies a column of larger section with reinforcement around 1-1.2 percent is more economical than a column of smaller section with 3-4 percent of reinforcement. Therefore, engineers should go for larger concrete section instead of larger percentage of reinforcement for optimum design.

If proper high strength of concrete can be obtained in field condition, it can result in minimizing the total cost of column. Using 5000 psi concrete instead of 3000 psi concrete saves 20-50 percent of total cost. Use of 1 percent of reinforcement sometimes increases the column dimension more than the acceptable limit. High strength concrete may be used in this case to reduce the section and at the same time it is economical too.

The circular column is more costly than a square column having same axial load and moment resisting capacity. However, sometimes engineers prefer circular column for architectural beauty or other practical purposes. But when cost is a vital factor, circular columns should be avoided as much as possible.

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REFERENCES

Nilson, A.H, Darwin, D. and Dolan, C.W. (2004). Design of Concrete Structures, 13th edition

Bangladesh National Building Code, BNBC 1993. Part 6 Chapter 2 Section 2.5, Earth Quake Loads Published by Housing and Building Research Institute and Bangladesh Standard and Testing Institute, Bangladesh.

Nevil, A.M. (1981). Properties of Concrete, 3rd Edition, Pitman Books Ltd, London,

Brester, B. "Design Criteria for Reinforced Columns under Axial Load and Biaxial Bending", J.ACI, Vol. 32. No. 5, 1960, PP 481-490,

Furlong, R.W. "Ultimate Strength of Square Column under Biaxially Eccentric Loads", J.ACI Vol. 32, No. 9, 1961, PP 1129- 1140

COMPARISON OF WIND LOAD AMONG BNBC AND OTHER CODES IN DIFFERENT TYPE OF AREAS

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ABSTRACT

Wind load as a part of lateral load is very important concern in structural analysis. Rationale wind load can be ensured in a structure by following established and tested building codes. In Bangladesh this purpose has been served by BNBC since 1993. The current BNBC 1993 is going to be replaced by an updated BNBC code which is popularly known as BNBC 2010. In this paper both BNBC codes are studied and compared between them as well as with NBC-India- 2005, IBC 2009 and ASCE 7-05 in terms of wind load using a parameter termed as factored total wind pressure. The investigation reveals that wind load in urban areas (Exposure A) according to BNBC 2010 is slightly higher than BNBC 1993. But wind load in obstructed and unobstructed open terrain type areas (Exposure B and C) according to BNBC 2010 is notable lower than BNBC 1993. Further wind load according to BNBC 2010 is exactly equal to wind load according ASCE 7-05 and slightly less than IBC 2009. Again, in urban and obstructed open terrain type areas NBC-India- 2005 is most conservative while BNBC 1993 is most conservative in unobstructed open terrain type areas in terms of wind load among the comparing codes.

Keywords: *BNBC, Other codes, Factored total wind pressure, Different type of areas, Increase or Decrease.*

1. INTRODUCTION

Bangladesh National Building Code (BNBC) was first organized in the year of 1993(Atique & Wadud, 2001). Afterwards initiative has been taken to update BNBC 1993 and a draft copy has already been prepared (BRTC & BUET,2010 ; Shafi, 2010). A total change at wind load provision in the proposed draft of the code are noticed. The new wind load provision in BNBC 2010 is an adaptation from ASCE 7-05. This proposed copy will be termed as BNBC 2010 in the rest portion of the discussing paper.

This paper aims at the comparison of provisions of wind load analysis given between BNBC 1993 and BNBC 2010. The designers who use BNBC 1993 as their basis to calculate the design wind load, this comparative study will provide them with a relation showing percent increase or decrease of design wind load in the new code with respect to the old one. Again, the comparison of BNBC with other building codes will inform how much factor of safety against wind disaster is imposed considering the economical aspects and population of our country.

2. METHODOLOGY

For wind pressure, BNBC 2010 has been compared with BNBC 1993, NBC-India- 2005 and IBC 2009. Basic features of these codes are presented in table 1. In all codes, calculation of design wind pressure is a two-step process. In BNBC 1993, first the sustained wind pressure is calculated on the basis of importance of structure, height and exposure condition and basic wind speed, which in turn depends on the region the structure is located in. The sustained wind pressure is then converted to design wind pressure by multiplication with the gust coefficient and pressure coefficient for the structure.

In BNBC 2010, first the sustained wind pressure is calculated on the basis of importance of structure, exposure and topographic condition of the region, directionality factor and wind basic wind speed, which in turn depends on the region where the structure is located. Finally, the design wind pressure is calculated by multiplying the sustained wind pressure with gust effect factor and external pressure co-efficient and adjusting the value for internal pressure. For both code, the exposure of the structure to wind forces is a function of terrain type, vegetation and built up environment in the surrounding and Pressure coefficient considers the direction of wind relative to the structure and roof slope.
In NBC-India-2005, the design wind speed at various heights is determined first on the basis of risk level, terrain roughness, local topography and geometry (height, size etc.) of the structure. The terrain factor refers to

exposure category. In addition another factor describes the local topography e.g. hills, valleys, cliffs, ridges etc. In the second step, design wind speed is converted to pressure by a simple conversion factor.

Table 10: Comparison of building codes with respect to wind force determination

BNBC 1993	BNBC 2010/ASCE 7-05	NBC 2005	IBC 2009
$q_z = C_C C_I C_Z V_b^2$ q_z = sustained wind pressure at height z, kN/m^2 C_c = velocity-to-pressure conversion coefficient = 47.2×10^{-6} C_I = structure importance coefficient C_z = combined height and exposure coefficient V_b = basic wind speed in km/hr (HBRI and BSTI, 1993:6-32)	$q_z = 0.000613 K_z K_d K_{zt} V^2 I$ q_z = velocity pressure at height z, kN/m^2 K_z = velocity pressure exposure coefficient K_d= wind directionality factor K_{zt}= topographic factor I = structural importance factor V= basic wind speed in m/s (HBRI and BRTC, 2010: Part 6-Chap 2) (ASCE/ SEI 7-05, 2006: chap 6-page 27)	$V_z = V_b k_1 k_2 k_3$ V_z = design wind speed at any height z in m/s V_b = basic wind speed in m/s k_1 = probability factor k_2 = terrain height and structure size factor k_3 = topography factor (BIS,2005: part 6- page 21)	$q_s = 0.00256\ V^2$ q_s=Wind stagnation pressure in psf. V= basic wind speed in mph. (INC, 2009:326)
$P_z = C_G C_P q_z$ p_z = design wind pressure at height z, kN/m^2 C_G = gust coefficient C_P=pressure coefficient for structures or components (HBRI and BSTI, 1993:6-34)	$P = qGC_p - q_i(GC_{pi})$ p = design wind pressure G= gust effect factor C_p = external pressure coefficient GC_{pi} =internal pressure coefficient (HBRI and BRTC, 2010: Part 6-Chap 2) (ASCE/ SEI 7-05, 2006: chap 6-page 28)	$p_z = 0.6\ V_z^2$ p_z = design wind pressure in N/m^2 at height z (BIS, 2005: part6- page 16)	$P_{net} = q_s K_z C_{net}[I K_{zt}]$ K_z=Velocity pressure exposure coefficient. C_{net}= Net-pressure coefficient. I=Importance factor. K_{zt}= Topographic factor. (INC, 2009:326)

In IBC 2009, first the stagnation pressure is determined from basic wind speed by a simple conversion factor. Then design wind pressures for buildings and structures is determined for any height on the basis of height, exposure and gust, direction of wind relative to structure, roof slope, importance of the structure and wind stagnation pressure.

3. ILLUSTRATIONS

3.1 Basic wind speed V & V_b

In comparing the basic wind speeds given between BNBC 1993 and BNBC 2010, it is important to note that BNBC 1993 specifies fastest-mile wind speeds whereas BNBC 2010 provides basic wind speed in terms of 3-second gust wind speeds. The fastest mile speed is the average speed of a particle traveling with the wind over the distance of one mile. The 3-second gust speed is the peak gust speed averaged over a short time interval of 3 seconds duration.

Both BNBC 1993 and BNBC 2010 provide basic wind speed associated with an annual probability of occurrence of 0.02 (50 year recurrence interval) measured at a point 33 ft (10m) above the mean ground level in a flat and open terrain. In both BNBC 1993 & BNBC 2010, tornadoes have not been considered in developing the basic wind speed distribution.

Table 11: Comparison of BNBC 1993 & BNBC 2010 with respect to basic wind speeds

LOCATION	BNBC 2010 Basic Wind Speed V (m/s)	BNBC 1993 Basic Wind Speed, V_b (km/hr)	BNBC 1993 Basic Wind Speed, V_b(m/s)	RATIO, V/V_b	RATIO^2, $(V/V_b)^2$	% INCREASE
Angarpota	47.8	150	41.67	1.15	1.32	31.61
Bagerhat	77.5	252	70.00	1.11	1.23	22.58
Bandarban	62.5	200	55.56	1.13	1.27	26.56
Barguna	80.0	260	72.22	1.11	1.23	22.70
Barisal	78.7	256	71.11	1.11	1.22	22.48
Bhola	69.5	225	62.50	1.11	1.24	23.65
Bogra	61.9	198	55.00	1.13	1.27	26.66
Brahmanbaria	56.7	180	50.00	1.13	1.29	28.60
Chandpur	50.6	160	44.44	1.14	1.30	29.62
Chapai Nawabganj	41.4	130	36.11	1.15	1.31	31.44
Chittagong	80.0	260	72.22	1.11	1.23	22.70
Chuadanga	61.9	198	55.00	1.13	1.27	26.66
Comilla	61.4	196	54.44	1.13	1.27	27.18
Cox's Bazar	80.0	260	72.22	1.11	1.23	22.70
Dahagram	47.8	150	41.67	1.15	1.32	31.61
Dhaka	65.7	210	58.33	1.13	1.27	26.85
Dinajpur	41.4	130	36.11	1.15	1.31	31.44
Faridpur	63.1	202	56.11	1.12	1.26	26.46
Feni	64.1	205	56.94	1.13	1.27	26.71
Gaibandha	65.6	210	58.33	1.12	1.26	26.47
Gazipur	66.5	215	59.72	1.11	1.24	23.99
Gopalganj	74.5	242	67.22	1.11	1.23	22.83
Habiganj	54.2	172	47.78	1.13	1.29	28.69
Hatiya	80.0	260	72.22	1.11	1.23	22.70
Ishurdi	69.5	225	62.50	1.11	1.24	23.65
Joypurhat	56.7	180	50.00	1.13	1.29	28.60
Jamalpur	56.7	180	50.00	1.13	1.29	28.60
Jessore	64.1	205	56.94	1.13	1.27	26.71
Jhalakati	80.0	260	72.22	1.11	1.23	22.70
Jhenaidah	65.0	208	57.78	1.13	1.27	26.56
Khagrachhari	56.7	180	50.00	1.13	1.29	28.60
Khulna	73.3	238	66.11	1.11	1.23	22.93
Kutubdia	80.0	260	72.22	1.11	1.23	22.70
Kishoreganj	64.7	207	57.50	1.13	1.27	26.61
Kurigram	65.6	210	58.33	1.12	1.26	26.47
Kushtia	66.9	215	59.72	1.12	1.25	25.48
Lakshmipur	51.2	162	45.00	1.14	1.29	29.45

Lalmonirhat	63.7	204	56.67	1.12	1.26	26.36
Madaripur	68.1	220	61.11	1.11	1.24	24.18
Magura	65.0	208	57.78	1.13	1.27	26.56
Manikganj	58.2	185	51.39	1.13	1.28	28.26
Meherpur	58.2	185	51.39	1.13	1.28	28.26
Maheshkhali	80.0	260	72.22	1.11	1.23	22.70
Moulvibazar	53.0	168	46.67	1.14	1.29	28.98
Munshiganj	57.1	184	51.11	1.12	1.25	24.81
Mymensingh	67.4	217	60.28	1.12	1.25	25.03
Naogaon	55.2	175	48.61	1.14	1.29	28.95
Narail	68.6	222	61.67	1.11	1.24	23.75
Narayanganj	61.1	195	54.17	1.13	1.27	27.24
Narsinghdi	59.7	190	52.78	1.13	1.28	27.95
Natore	61.9	198	55.00	1.13	1.27	26.66
Netrokona	65.6	210	58.33	1.12	1.26	26.47
Nilphamari	44.7	140	38.89	1.15	1.32	32.12
Noakhali	57.1	184	51.11	1.12	1.25	24.81
Pabna	63.1	202	56.11	1.12	1.26	26.46
Panchagarh	41.4	130	36.11	1.15	1.31	31.44
Patuakhali	80.0	260	72.22	1.11	1.23	22.70
Pirojpur	80.0	260	72.22	1.11	1.23	22.70
Rajbari	59.1	188	52.22	1.13	1.28	28.07
Rajshahi	49.2	155	43.06	1.14	1.31	30.58
Rangamati	56.7	180	50.00	1.13	1.29	28.60
Rangpur	65.3	209	58.06	1.12	1.27	26.51
Satkhira	57.6	183	50.83	1.13	1.28	28.39
Shariatpur	61.9	198	55.00	1.13	1.27	26.66
Sherpur	62.5	200	55.56	1.13	1.27	26.56
Sirajganj	50.6	160	44.44	1.14	1.30	29.62
Srimangal	50.6	160	44.44	1.14	1.30	29.62
St. Martin's Island	80.0	260	72.22	1.11	1.23	22.70
Sunamganj	61.1	195	54.17	1.13	1.27	27.24
Sylhet	61.1	195	54.17	1.13	1.27	27.24
Sandwip	80.0	260	72.22	1.11	1.23	22.70
Tangail	50.6	160	44.44	1.14	1.30	29.62
Teknaf	80.0	260	72.22	1.11	1.23	22.70
Thakurgaon	41.4	130	36.11	1.15	1.31	31.44
					Average	26.58

Since square of the basic wind speed is used in determining sustained wind pressure, the increased wind speed results in approximately 26.58 percent increase in sustained wind pressure. Following equation is found satisfactory for converting fastest mile per hour wind speed into three second gust wind speed and used later for comparison –

$$V_{3s}=0.2986*V_{fmph}+2.986 \quad (1)$$

Where,
V_{3s}= three second gust wind speed in m/s
V_{fmph}= Fastest mile per hour wind speed in Km/hr ; (Faysal, 2013:Chap 4-Page 15)
(INC, 2009: Chap16-page 319)

3.2 Factored total wind pressure comparison of different codes

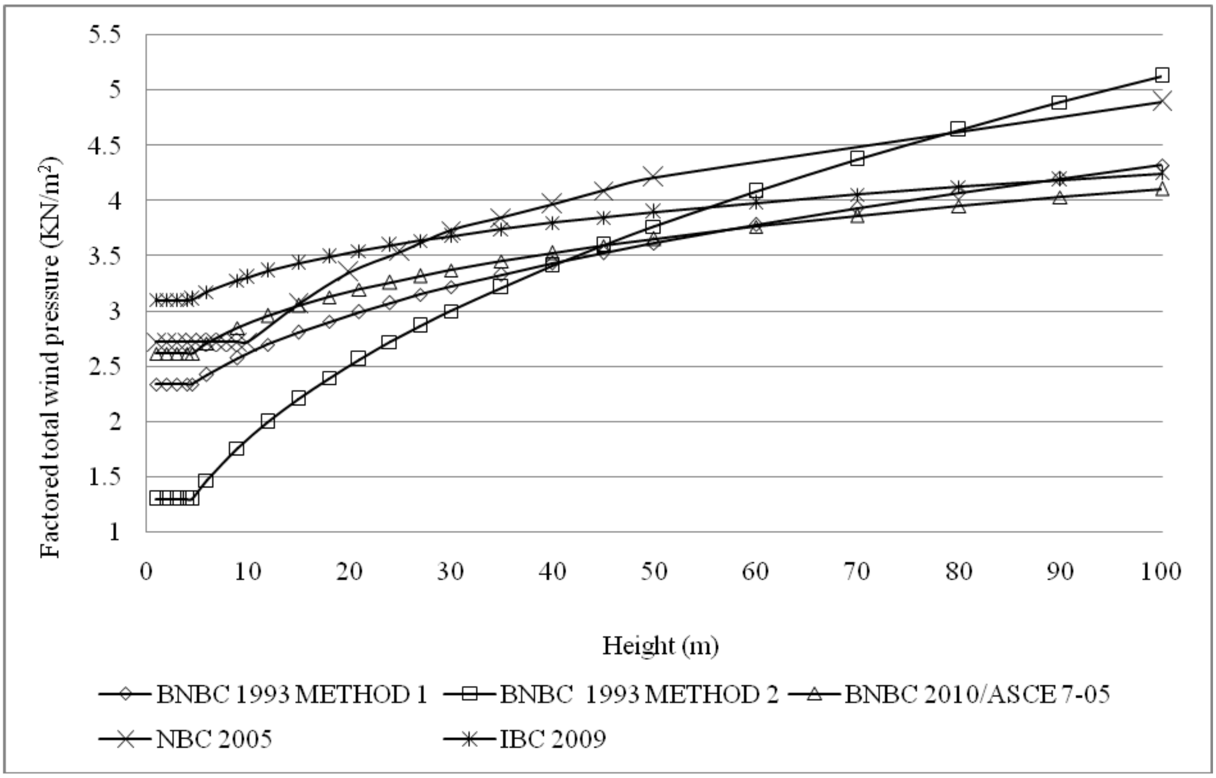
The basic wind speed is determined on the basis of three second gust speed for all the discussed codes except for BNBC 1993. BNBC 1993 specifies basic wind speed on the basis of fastest-mile wind speed. So, for comparison wind speed of 180 km/hr has taken for BNBC 1993 which is equivalent to 3s gust of 56.70 m/s (see, equation 1). For other codes, a velocity of 56.70 m/s has chosen. ASCE 7-05 is not compared separately as proposed BNBC 2010 gives factored total wind pressure exactly same to ASCE 7-05. So, ASCE 7-05 and BNBC 2010 can be used interchangeably.

The effect of surrounding objects and height of structures are considered through various parameters in different building codes such as class, Terrain category, Exposure category etc. These parameters are not same for all codes. So, for comparison purpose surrounding conditions are broadly classified into three categories. They are urban, obstructed open terrain and unobstructed open terrain type areas which are defined in BNBC as exposure A, B and C respectively. All the codes are compared for these major surrounding conditions. For only BNBC 1993 instead of single method, two alternative methods are presented. In the Method 1, the windward and the leeward pressure are separately considered and then combined. But in Method 2, the overall pressure coefficient is used to determine the design wind pressure directly. Finally, all pressures are multiplied with respective wind load factor to calculate factored total wind pressure The following pressure variation is typical for structures of 100 m height. Both length and width of the structures have taken equal to 20 m. Corresponding parameters used in the respective codes are presented in the following table and the comparison results are presented graphically in Graph 1, Graph 2 and Graph 3.

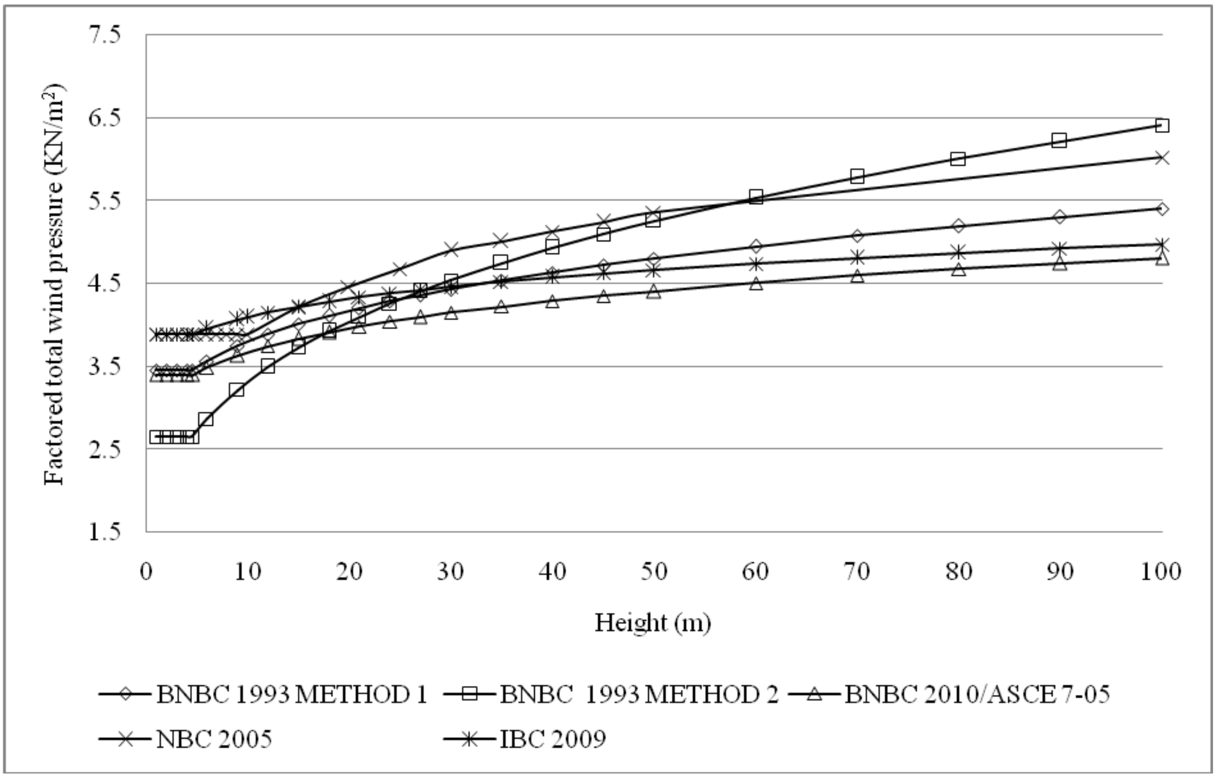
Table 12 : Data used in Graph 1, 2 and 3.

		BNBC 1993 Method 1	BNBC 1993 Method 2	BNBC 2010/ ASCE 7-05	NBC 2005	IBC 2009
Basic Wind speed	Graph 1	180 km/hr[1]	180 km/hr[1]	56.70 m/s	56.70 m/s	128.86 mph[2]
	Graph 2	180 km/hr[1]	180 km/hr[1]	56.70 m/s	56.70 m/s	128.86 mph[2]
	Graph 3	180 km/hr[1]	180 km/hr[1]	56.70 m/s	56.70 m/s	128.86 mph[2]
Terrain Category (BIS, 2001-chap-3)	Graph 1	-	-	-	3	-
	Graph 2	-	-	-	2	-
	Graph 3	-	-	-	1	-
Class	Graph 1	-	-	-	C	-
	Graph 2	-	-	-	C	-
	Graph 3	-	-	-	C	-
Exposure Category	Graph 1	A	A	A	-	B
	Graph 2	B	B	B	-	C
	Graph 3	C	C	C	-	D
Building type	Graph 1	-	-	Enclosed	-	Enclosed
	Graph 2	-	-	Enclosed	-	Enclosed
	Graph 3	-	-	Enclosed	-	Enclosed
Wind load Multiplying factor	All Graphs	1.3	1.3	1.6	1.5	1.6

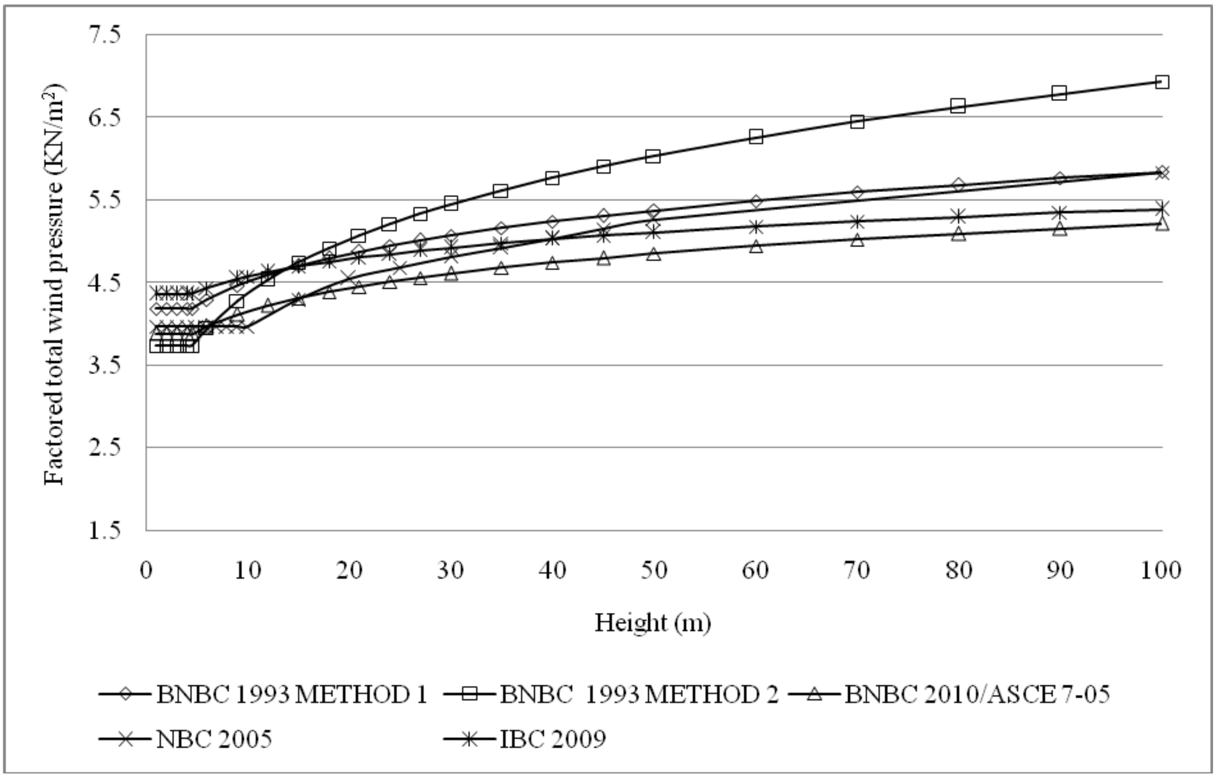
[1] Equivalent wind speed for 3 s gust of 56.70 m/s
[2] Equivalent to 56.70 m/s ; [3] (-) indicates not applicable in corresponding code



Graph 1: Factored total Wind Pressure comparison of different codes for urban areas.



Graph 2: Factored total wind Pressure comparison of different codes for obstructed open terrain type areas.



Graph 3: Factored total Wind Pressure comparison of different codes for unobstructed open terrain type areas.

4. CONCLUSIONS

Before BNBC 1993 a simple empirical formula is used to determine wind load which do not consider effect of surrounding objects and height of structures in wind pressure. This shortcoming has overcome in BNBC 1993 by introduction of concept of exposure category and gust factor. The effect of surrounding objects and height of structures is further upgraded in proposed BNBC 2010 wind provision. As a result Wind load in urban areas (Exposure A) according to BNBC 2010 is found considerably higher (7-12 %) than BNBC 1993. But wind load in obstructed and unobstructed open terrain type areas (Exposure B and C) according to BNBC 2010 is found significant lower (2-10 %) than BNBC 1993. Further wind load according to BNBC 2010 is found exactly equal to wind load according ASCE 7-05 and considerably less (7-18 %) than IBC 2009. Again, the wind load according to Indian NBC 2005 wind provision gives the highest wind load among all comparing codes in urban and obstructed open terrain type areas. In obstructed open terrain type areas BNBC 1993 gives highest wind load.

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REFERENCES

ASCE/SEI 7-05, 2006. *A.S.C.E. Minimum design loads for buildings and other structures*, Reston, Virginia.

Atique, F. & Wadud, J. 2001. *A comparison of BNBC-93 with other building codes with respect to earthquake and wind analysis.* The Eighth East Asia-Pacific Conference on Structural Engineering and Construction, Nanyang Technological University, Singapore.

BIS, 2001. *Indian Standard Code of Practice for design loads (other than earthquake) for buildings and structures. Part 3 – Wind loads, IS: 875 (Part 3) -1987,* New Delhi.

BIS, 2005. *National Building Code of India,* New Delhi.

BRTC and BUET, 2010.*Updating of Bangladesh National Building Code,*Inspection Report submittted to HBRI. Retrived from -http://www.buet.ac.bd/ce/pdf/Top%20pages_Inception%20Report.pdf

Faysal, R.M. 2013 . *A comparative study on Wind Load analysis of Bangladesh National Building Code (BNBC) with other building codes* .Undergraduate thesis, Faculty of civil engineering, BUET.
HBRI and BSTI, 1993. *Bangladesh National Building Code*, Dhaka.
HBRI and BRTC, 2010. *Bangladesh National Building Code(draft)*, Dhaka.
INC, 2009. *International Building Code,* California.
Shafi, S. A. 2010. *Keynote Paper on National Building Code and Its Implementation*, Round Table Discussion on Implementation of National Building Code, Press Club, Dhaka. Retrived from-
http://www.blast.org.bd/content/key-note/national-building-code-29June.pdf

AN ALTERNATE TO SULFUR CAPPING FOR CRUSHING STRENGTH TEST OF CLAY BRICKS IN BANGLADESH

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ABSTRACT

Kiln-burnt clay bricks are usually produced with improper surface flatness; however, accurate determination of crushing strength demands that the load bearing surfacesof clay bricks should be flat during testing, through a suitable capping system, so that the load is transferred uniformly over the entire brick sample. In capping system, sulfur capping and gypsum capping are commonly used for bricks in accordance with the ASTM standard. However, sulfur imposes significant health hazard and gypsum capping are relatively expensive in context of Bangladesh. Developing an alternate capping system that could eliminate the potential problemsassociated with brick capping for crushing strength determinationof brick has become a prime necessity for the material engineers. This research investigated few capping systems and found that capping with mortar was a possible solution if it could produce the similar test results as by the other standard methods. Experimental results showed that a cement:sand ratio of 1:0.75 with 22% water of total material could produce similar test results as of the sulfur-filler capping after two days curing.

Keywords: Clay brick, Crushing strength, Sulfur Capping, Gypsum Capping.

1. INTRODUCTION

Bricks are one of the major construction materials for low-rise building construction. Approximately 8.6 billionbricks are produced annually in Bangladesh (UNDP, 2011).Whole brick units are preferable over ready-made building blocks due to their relatively lower cost. Their usages range from load-bearing vertical walls to earth retaining masonry structures. Also, brick-chips are preferable as the coarse aggregate over stone-chips where lower dead load and moderate concrete strength is required. In a properly designed concrete mix,the plane of failure is not generally expected to pass through the aggregates alone. Therefore, the mechanical capacity of bricks, either as a whole unit or as coarse aggregates, should be known in advance before their placement as a structural component.

Bricks are generally molded manually with cohesive clay and burnt in the coal-burn or wood-burn kilns. The operating temperatures in the kiln are mostly controlled on approximation (IBSTOCK, 2005) and very often result in improper surface flatness of the burnt bricks. However, accurate determination of the crushing strength of the bricks demands that the surface which wouldbecome bearing surface during compressive strength test should be plain so that the load would betransferred uniformly over the whole bearing surface(Oxyildirim and Carino, 2006).This process is known as capping.ASTM standards specify two capping systems, namely, sulfur filler capping and gypsum capping for crushing strength determination of structural clay bricks (ASTM, 2005). Among the two, the sulfur filler capping is usually followedin Bangladesh. The sulfur mortar consists of 55-70 weight percent of sulfur and 30-45 weight percent of silicious flour (Gilson, 2014). The standard procedure in sulfur filler capping involves a thermostaically controlled heating pot to prevent overheating of the molten sulfur and to ensure sufficient fluidity to be available for a reasonable period of time (ASTM, 2005). Safety precautions during sulfur melting strongly recommend the sulfur vapour to be trapped with a strong hood.However, the current practice of sulfur filler capping in Bangladesh imposes serious technical and safety issues. For melting sulfur, open electric heateris commonly used, where there is hardly any provision for temperature controlandsulfur steam emanations may set on fire at excessive heating.During positioning the bricks on molten sulfur, due to self weightof bricks the capping thickness may not be uniform throughout the loading surface, which may invalidate the purpose of capping and significantly affect the test results (Kelch and Emme, 1958). The sulfur vapor, released during the melting process, is highly toxic.Continuous exposure may cause extreme respiratory distress and eye and skin irriation (Gilson, 2014).The intense odour of sulfur vapor creates an uncomfortable working environment in the neighborhood of the open heater for a significantly long period of time. In order to avoid these sort of problems, an alternate capping system for bricks using cementmortar has been in practice in some material testing laboratories (CRTS, 2014). While it was able to overcome the safety issues, absence of having any standard code in regards to that practice, the true capacity of the bricks, determined through this capping system, is questionable. In addition, hardly any literature can be found on the cement mortar capping system of bricks. Thus, its relationship with the ASTM standard capping system is still unknown.

The purpose of the present study is to develop an alternate capping system using cement mortar for crushing strength determination of clay bricks and to establish a relationship between the cement mortar capping and the standard ASTM sulfur filler capping.

2. EXPERIMENTAL PROGRAM

2.1 Materials

The sulfur which was used for capping bricks contained 60 weight percent of sulfur as a mixture where the remainder being ground fire clay passing a no. 100 sieve without plasticizer (ASTM, 2005). Its melting point was 119ºC.

River sand was used as the fine aggregate for making mortar. Fineness modulus (FM) of sand is an important factor for mortar strength.In order to avoid the variation in FM in different local sands, a FM of 1.5 was maintained throughout the experiments according to the (AASHTO, 2006). The FM was confirmed by mixing two local sands having FM of 1.05 and 1.68, and by using equation (1). The grain size distribution of the local sands and the prepared sandis shown in Figure 1.

$$F_{com} \times m_{total} = (m_1 F_1 + m_2 F_2) \quad (1)$$

Where,
F_{com}=FM of prepared sand(1.50)
F_1=FM of Kushtia sand (1.68)
F_2 = FM offine sand (1.05)
m_1=Mass of Kushtia sand
m_2=Mass of fine sand
m_{total}= Total mass $= m_1 + m_2$

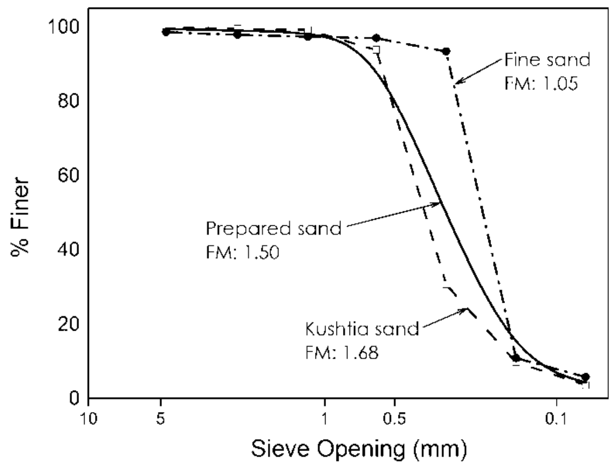

Figure 18: Grain size distribution of fine aggregate

Portland composite cement was used as the cementitious binder. Its compressive strength at 3 and 7 days was 3640 psi and 4280 psi, respectively.First class clay bricks from five different producers were used for the entire investigation. The average absorption capacity of the bricks in 24 h was 16.5%.

Table 13: Mortar mix proportion for frog mark depression

Batch	Materials (kg/m^3)			Cement: Sand: Water (by mass)	Curing duration (day)
	Cement	Sand	Water		
B1	670	1345	270	1 : 2 : 0.4	2
B2	775	1160	310	1 : 1.5 : 0.4	2
B3	905	905	365	1 : 1 : 0.4	2
B4	995	745	400	1 : 0.75 : 0.4	2
B5	1100	550	440	1 : 0.5 : 0.4	2
B6	995	745	400	1 : 0.75 : 0.4	3
B7	1100	550	440	1 : 0.5 : 0.4	3

2.2 Sample preparation

At first, ten bricks of a selected brand (Super brick) were taken and three cubes (2"×2"×2") were sawn from each brick. Then average compressive strength of the cubes was determined. By this way an approximate brick strength was obtained. Before proceeding for the capping layer, each brick was divided into two halves by a concrete cutter and the dimensions were measured. The frog-mark depression of the brick samples so formed was then filled with mortar made of Portland composite cement and cured for a specified length of time. The mix proportion of the mortar was determined on trial basis (Table 1) so thatit would provide the same strength as of the brick cubes. So, it was expected that after filling and levelling the depression with the mortarof desired mix ratio, the whole brick unit would show homogeneity in terms of crushing strength. The capping process was then executed.

2.2.1 Procedure for Sulfur capping

Sulfur capping was performed following the ASTM standard to serve as the reference samples (ASTM, 2005).A rectangular mold of size 5.5 in. x 5.5 in. was used for sulfur capping, where the casting surface plate was closed by four steel bars of 0.25 in. thickness, as shown in Figure 2. Melting sulfurwas then poured in the mold with a depth of 0.25 in. and the half-sized brick sample was place on it after 5 min. and kept there for another 5 min. A thin coating of mineral oil was used beforehand on the capping plates to prevent bonding between the plates &capping material.The opposite side of the brick sample was capped in the same way. A typical brick sample right after sulfure capping is shown in Figure 3. Compressive strength test was performed 2h after sulfur capping. At least ten brick sampleswere tested from each batch.

Figure 19: Mold for sulfur capping

Figure 20: Brick sample right after Sulfur capping

2.2.2 Procedure for mortar capping

The mortar cappingwas prepared with six mix proportion, shown in Table 2. Different brand bricks were used for capping. The cement content varied from 305 to 905 kg/m^3 and the w/c varied from 0.4 to 1. A mortar layer of 0.25 in thick was laid on one 5 in. x 5 in. load bearing surface of half brick samples and properly levelled. After 24 h of moist curing at 23^{0}C and 65% RH, the opposite face of the brick samples were capped and levelled in a similar way. Crushing strength test was performed after another 24 h or 48 h of moist curing. Brick samples with typical mortar capping before crushingare shown in Figure 4.

Table 14: Mix design for mortar capping (Random Selection)

Batch	Materials (kg/m^3)			Cement: Sand: Water (by mass)
	Cement	Sand	Water	
C1	350	1400	350	1 : 4 : 1
C2	370	1385	350	1 : 3.75 : 0.95
C3	505	1265	355	1 : 2.5 : 0.70
C4	595	1185	355	1 : 2 : 0.60
C5	720	1075	360	1 : 1.5 : 0.50
C6	905	905	365	1 : 1 : 0.40

Figure 21: Typical mortar capping

To overcome the problem of the variation of the strength due to the inner property of brick sample, eight bricks were selected for each batch and divided into two parts. One part was tested for sulfur capping and the other part for mortar capping. Mortar capping was prepared with three mix proportion and 22% water content of the total material according to the above stated procedure. Bricks were capped one part of the specimens with the mortar and age them for two days with closed environment. Batch D1 was tested without oven dry condition and the other specimens were kept 24 h at 110˚C in oven(Table 3).

Table 3: Mix design for mortar capping(Identical Brick)

Batch	Materials (kg/m^3)			Cement: Sand: Water (by mass)	Drying Condition
	Cement	Sand	Water		
D1	515	1035	440	1 : 2 : 0.85	Without oven dry
D2	515	1035	440	1 : 2 : 0.85	24h oven dry
D3	790	790	435	1 : 1 : 0.55	
D4	900	685	445	1 : 0.75 : 0.50	

3. RESULTS AND DISCUSSION

Approximate crushing strength of bricks was determined from ten bricks of a selected brand, by sawing 3 numbers of 2-inch cubes from each brick. The test results are shown in Figure 5.For different mix proportion shown in Table 1, average compressive strength of mortar cube was calculated (Figure 6) for filling the depression of frogmark.

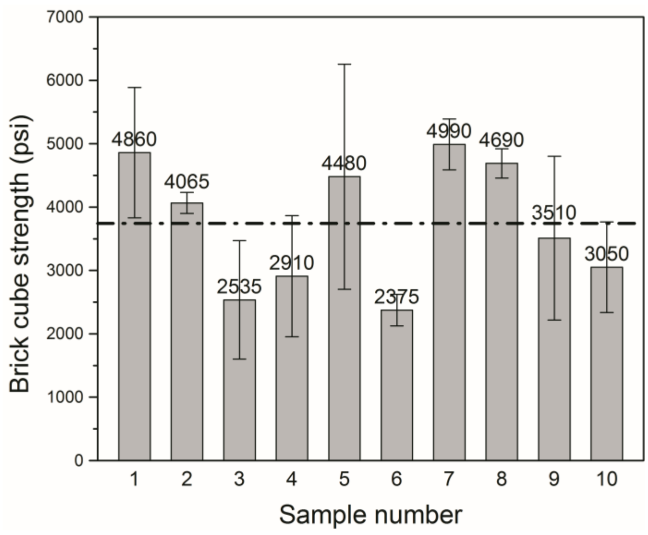

Figure 5: Approximate crushing strength of brick from brick-cube test

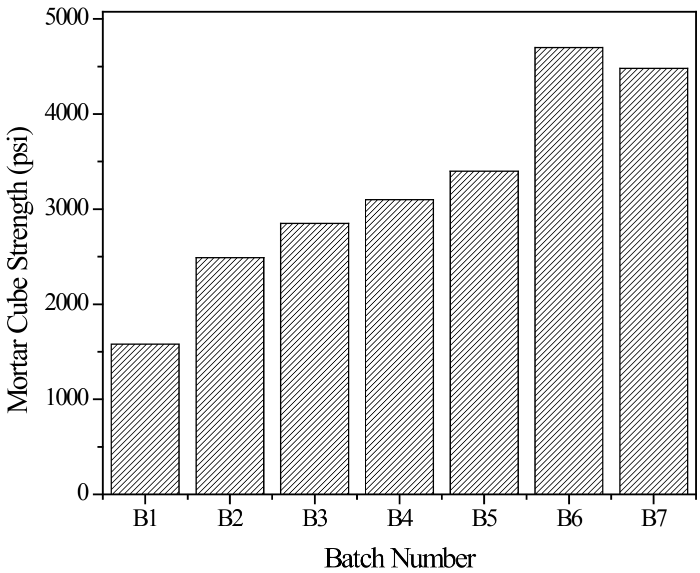

Figure 6: Average compressive strength of mortar for different mix proportion

The approximatemean compressive strength found from 30 brick cubes was 3750 psi.Again from Figure6, approximately equalaveragecompressive strength of mortar 3400 psi wasselected for batch B5. This mortar mix proportion was used to fillup the depression of frogmark as an alternative of quick-hardening cement(ASTM, 2005).

The average compressive strength of brick samples for different mortar mix proportion were shown in Figure 7. It can be seen that general practiced method of mortar capping can not evaluate the proper strength of brick as it varies significantly from the standard sulfur capping. On the other hand, any relationship between the capping systems can not be developed due to random selection of brick samples. Moreover, all bricks in a specific brand will not be uniform.

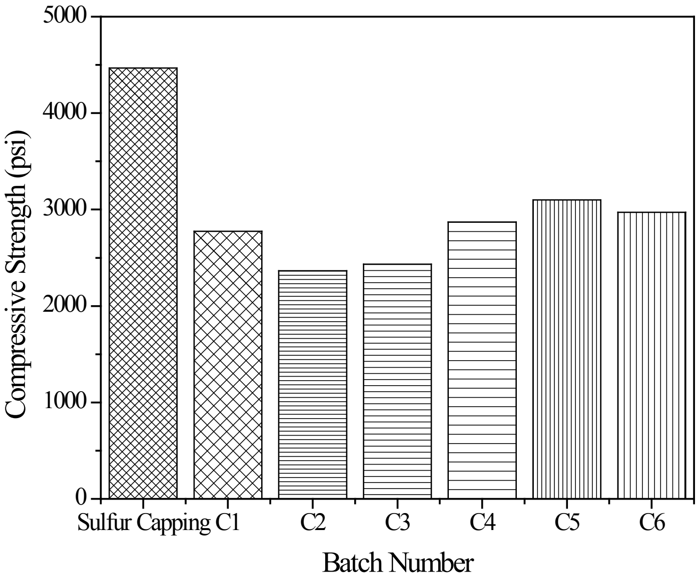

Figure 7: Comparison of compressive strength of brick in between sulfur and mortar capping

Based on the obtained results, a correlation between the strength of identical bricks for the two different capping conditions were established (Figure 8). The linear regression was considered to best fit the data as indicated by the curve equation relating each set of variables. In addition, each curve equation is presented by its regression fitting (R2).

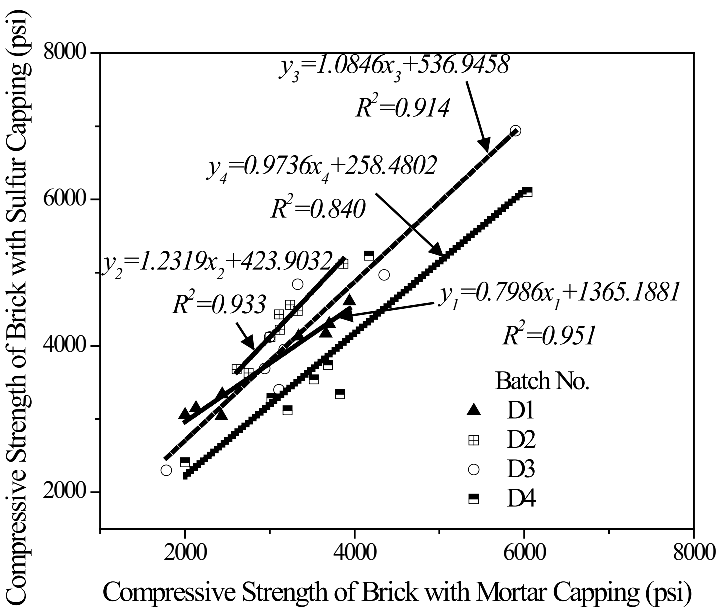

Figure 8: Relationship of the compressive strength of brick having mortar capping and sulfur capping

In the analysis of the relation, it is clear that linear regression can best describe this relation. It can be seen that the values of compressive strength for sulfur capping are generally higher than that of the compressive strength of mortar capping.The linear equation drawn for each brick with different mortar ratio and various oven dry conditions can be used to make this conversion as shown in Figure 8.It can also be described that with the exception of the 'without-oven-dry' condition as amount of cement decreases, the difference in compressive strength between mortar capping, and sulfur capping increases.

In order to have an assessment of the relationship between sulfur and mortar capping forbatch D4, three different brand bricks were sampled only for batch D4 havingmortar ratio 1:0.75 (cement:sand)by following the same procedure.

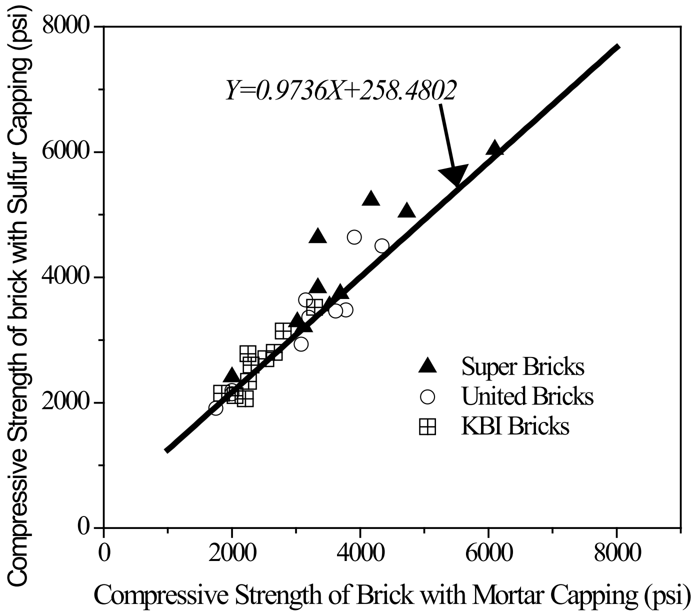

Figure 9: Validation of the equation with different brand bricks for Batch D4

The figure shows that data points for different brand bricks were clustered around the line obtained from the regression analysis. This indicates thatY=0.9736+258.4802 is valid for all types of bricks with mortar ratio 1:0.75:0.50(cement:sand:water), two days capping age with closed environment, and 24h oven dry. So this can be suggested as an alternate procedure for brick capping.

4. CONCLUSIONS

An alternate capping system using ordinary Portland cement mortar for crushing strength determination of clay bricks was investigated. Before execution of capping, whole brick unit was made homogeneous in terms of crushing strength. Random capping specimens were identified with a large dissimilarity of obtained results. Therefore, identical bricks were used for both sulfur and mortar capping. Bricks of different brands were tested forsubstantiation. Based on the obtained data and the analysis above, a summary of findings are listed below:

1. After filling and levelling the frog-mark depression with a cement mortar of mix ratio 1:0.5:0.4 (cement:sand:water) with 2 days curing, the whole brick unit would show homogeneity in terms of average crushing strength.
2. The values of compressive strength withsulfur capping were generally higher than that with mortar capping. Linear regression was considered to best fit the data.
3. A mortar capping of cement-to-sand ratio of 0.75 and w/c of 0.50 was found to produce a crushing strength of the clay brick specimens with close approximation of the standard sulfur capping.
4. The actual crushing strength (in psi) with sulfur capping was found to be 0.9736 times the crushing strength with mortar capping plus about 258.5 psi.
5. The developed relationship between mortar and sulfur capping needs further verification in regards to the standard gypsum capping.

REFERENCES

ASTM C 67 (2005).“Test Methods of Sampling and Testing Brick and Structural Clay Tile”, Annual book of ASTM Standards, Vol.04.05, American Society for Testing Materials, West Conshohocken, PA, USA.

AASHTO T27 (2006).“Sieve Analysis of Fine and Course aggregates”,American Association of State Highway and Transportation Officials, Washington, DC, USA.

ASTM C 109 (2005).“Test Method for Compressive Strength of Hydraulic cement mortars”, Annual Book of ASTM Standards, Vol.04.01, American Society for Testing Materials, West Conshohocken, PA, USA.

CRTS (2014).Consultancy, research & testing services, Dept. of Civil Engineering, KUET, Khulna, Bangladesh.

Gilson Inc., Capping compound, Material safety data sheet, Gilson company inc., Valdosta, Georgia, USA, accessed: January 2014, http://www.globalgilson.com/documents/msds/hm-184.pdf

IBSTOCK (2005). “How Clay Bricks are Made”, Technical Information Sheet 16, Issue1, accessed: January 2014, http://www.ibstock.com/pdfs/technical-support/TIS16Howbricksaremade.pdf

Kelch, N.W. andEmme, F.E. (1958). “Effect of Type, Thickness, and Age of Capping Compounds onthe Apparent Compressive Strength of Brick”, ASTM Bulletin 230, May, pp. 38-41.

Oxyildirim, C. and Carino, N.J. (2006).“Concrete Strength Testing,” Significance on Test and Properties of Concrete and Concrete-Making Materials, ASTM International, West Conshohocken, PA, pp. 125-140.

UNDP (2011).“Eco-friendly brick technique helps build a cleaner Bangladesh”, accessed: January 2014,http://www.undp.org/content/undp/en/home/presscenter/articles/2011/06/20/eco-friendly-brick-technique-helps-build-a-cleaner-bangladesh/

COMPARISON OF THE COST OF CIRCULAR COLUMN WITH RECTANGULAR COLUMN FOR SIMILAR LOADING CONDITION

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ABSTRACT

In designing a building we often provide circular column .We provide a circular section as per architectural design, we don't think of the cost. We must have a knowledge between the cost of a circular column and rectangular column. Here we analyzed The cost of various circular column and square column having same axial load and moment resisting capacity. The required cost for a circular column is 20-60 percent more than a square column having same axial load and moment resisting capacity.

Keywords: Circular column, rectangular column, comparison of cost.

1. INTRODUCTION

Generally we design our structure according to architectural design. We randomly use circular column. But rare we compare the required cost for a circular column instead of using a square column. Here we will analyse the cost of circular column and then we will compare the cost of square column and circular column.

2. METHODOLOGY

First of all we need to develop a relation between a circular column and a rectangular one.
To develop a relation between a circular column and a rectangular column here we follow two approach:

1. Comparing the axial load capacity and moment resisting capacity between a circular column and a rectangular column having a equal gross concrete area and reinforcement.
2. Design a circular column and rectangular column for a specific load and moment and compare the required cost .

2.1 Design Method

To design column we used PCA COL software. Here the code ACI 318-89 used to design the column section.

2.2 Selection of Column

We selected here 8 column samples. For a specific loading condition we designed a square column. For the same loading condition we designed a circular column:
-Keeping the gross area same but changing the required reinforcement
-keeping the reinforcement same but changing the required gross area.

2.3 Analysis of Cost

Here we analyzed the required cost for a 10 ft column (Ground floor) for different condition.
Here for simplification we only use 60 grade reinforcement and 4000 psi concrete.
Cost of 60 grade reinforcement = 600000 BDT /ton;
Cost of concrete (4000psi) = 210 BDT/cft

2.4 Analysing the Cost Changing the Concrete Area

We selected 8 sample of column and the obtained result is below:

Table 1 : Analysing the Cost Changing the Concrete Area

SAMPLE	L (in)	B (in)	Acon (in2)	No of Bar	Ast (in2)	Percentage of steel	Axial load capacity (kip)	Moment capacity Mx (kip-ft)	Moment capacity My (kip-ft)	Cost of Concrete (BDT)	Cost of Reinforcement (BDT)	Total Cost (BDT)
A	12	12	144	8#6	3.52	2.44	128	73	73	2100	3267	5367
	Dia	16.75	220.5	8#6	3.52	1.60	128	74	74	3216	3267	6482
B	15	15	225	8#8	6.32	2.81	220	164	164	3281	5865	9146
	Dia	21.25	355	8#8	6.32	1.78	220	166	166	5177	5865	11042
C	16	16	256	8#8	6.32	2.47	252	190	190	3733	5865	9598
	Dia	22.75	406.5	8#8	6.32	1.55	252	194	194	5928	5865	11793
D	19	19	361	8#9	8	2.22	369	312	312	5265	7424	12689
	Dia	27	572.5	8#9	8	1.40	366	316	316	8349	7424	15773
E	20	20	400	8#9	8	2	413	350	350	5833	7424	13258
	Dia	28.5	638	8#9	8	2.01	413	359	359	9304	7424	16728
F	22	22	484	8#10	10.16	2.1	505	481	481	7058	9429	16487
	Dia	31.25	767	8#10	10.16	2.07	499	488	488	11185	9429	20614
G	24	24	576	8#10	10.16	1.76	611	585	585	8400	9429	17829
	Dia	34	908	8#10	10.16	1.77	602	595	595	13242	9429	22670
H	25	25	625	8#10	10.16	1.63	657	643	643	9115	9429	18543
	Dia	35.5	990	8#10	10.16	1.65	648	657	657	14438	9429	23866

Considering the change in gross concrete are only It is seen that for a same loading condition a circular cilumn required more concrete area than a square column. It means for a same axial load and moment resisting capacity a circular column is costs more than a square column. About 20-30 percent more.

2.5 b Analysing the Cost Changing the Reinforcement

We selected 8 sample of column and the obtained result is below:

Table 2 : Analysing the Cost Changing the Reinforcement

SAMPLE	L (in)	B (in)	Acon (in2)	No of Bar	Ast (in2)	Percentage of steel	Axial load capacity (kip)	Moment capacity Mx (kip-ft)	Moment capacity My (kip-ft)	Cost of Concrete (BDT)	Cost of Reinforcement (BDT)	Total Cost (BDT)
A	12	12	144	8#6	3.52	2.44	128	73	73	2100	3267	5367
	Dia	13.5	143.14	8#10	10.16	7.10	132	74	74	2087	9429	11516
B	15	15	225	8#8	6.32	2.81	220	164	164	3281	5865	9146
	Dia	17	226.98	8#14	18	7.93	231	170	170	3310	16705	20015
C	16	16	256	8#8	6.32	2.47	252	190	190	3733	5865	9598
	Dia	18	254.47	11#11	17.16	6.74	258	193	193	3711	15925	19636
D	19	19	361	8#9	8	2.22	369	312	312	5265	7424	12689
	Dia	21.5	363.055	15#10	19.05	5.25	372	310	310	5295	17679	22974
E	20	20	400	8#9	8	2	413	350	350	5833	7424	13258
	Dia	28.5	397.61	16#10	20.32	5.11	420	352	352	5798	18858	24656
F	22	22	484	8#10	10.16	2.1	505	481	481	7058	9429	16487
	Dia	25	490.87	11#14	24.75	5.04	504	481	481	7159	22969	30127
G	24	24	576	8#10	10.16	1.76	611	585	585	8400	9429	17829
	Dia	27	572.56	12#14	27	4.72	613	587	587	8350	25057	33407
H	25	25	625	8#10	10.16	1.63	657	643	643	9115	9429	18543
	Dia	28	615.75	13#14	29.25	4.75	671	664	664	8980	27145	36125

Considering the change in reinforcement only It is seen that for a same loading condition a circular cilumn required more concrete area than a square column. It means for a same axial load and moment resisting capacity a circular column is costs more than a square column.about 80- 130 percent more.

From the above analysis a graph like below is obtained.

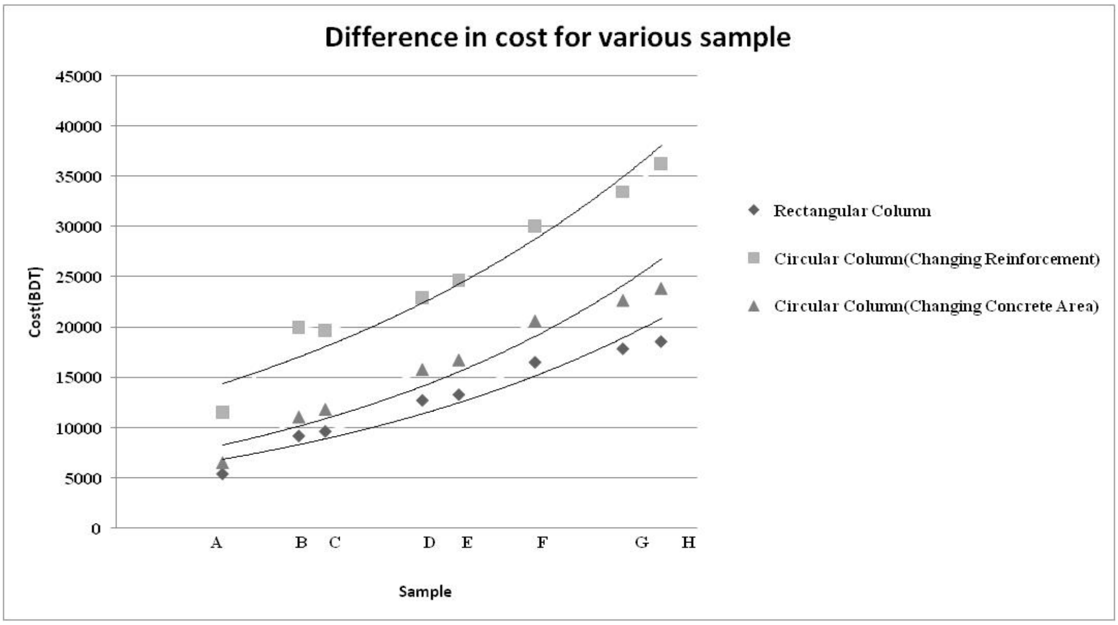

Figure 1: Difference in cost for various sample.

From the above graph a simple graph to compare the cost of circular column to square column can be obtained like below:

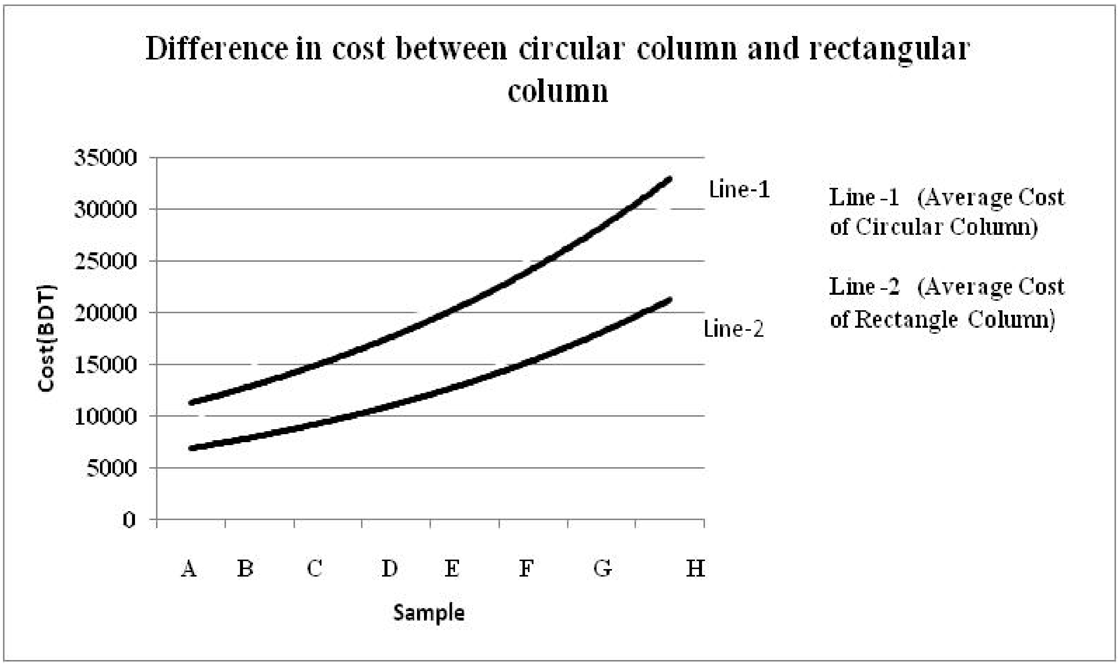

Figure 2: Difference in cost between circular column and rectangular column.

Form the above graph it is seen that the required amount of reinforcement and concrete area increase for a circular column for a specific loading condition. The required cost for a circular column is 20-60 percent more than a square column having same axial load and moment resisting capacity.

3. CONCLUSIONS

A square column may be used to reduce the total cost of column of a building. However sometime it is not possible to provide square column for architectural beauty or some practical purposes. So we have to provide circular column. But when cost is a vital factor then we must avoid providing circular column randomly.

REFERENCES

Nilson, A.H., Darwin, D. and Dolan, C.W.(2004), "Design of Concrete Structures", 13th ed.,

R.W Furlong, "Ultimate strength of square columns under Biaxially Ecentric Loads, "J,ACI vol 32, no 9,1961,pp 1129- 1140.

Nevil, A.M., "Properties of Concrete", 3rd Edition, Pitman Books Ltd., London, 1981.

CSI ETABS 9.7 software.

PCA COL software.

B.Bresler, "Design Criteria for Reinforced Columns under Axial load and Biaxial Bending ."J.ACI,vol.32.no 5,1960,pp 481-490.

Bangladesh National Building Code(BNBC),1993.

SELF-COMPACTING CONCRETE USING BARAPUKURIA FLY ASH

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ABSTRACT

Self-compacting concrete (SCC) is a high-performance concrete that can flow by its own weight to completely fill the formwork and self-consolidates without any mechanical vibration (compaction). Such concrete enhances performance and economy. This also helps environment through overcoming inaccessibility of compaction, want of equipment or skilled labor etc. Successful production of SCC depends upon an appropriate balance between the yield stress and viscosity of the paste by providing high range water reducers (HRWR). Considering the environmental impact in cement production, the world is moving towards supplementary cementitious materials such as fly ash, GGBS, silica fume and metakaolin for sustainable construction. This study aimed to produce SCC using Barapukuria (Bangladesh) fly ash. In this regard concrete samples were produced using Ordinary Portland Cement (OPC; Control) and fly ash level of 10, 20 and 30%. Locally available chemical admixtures (HRWR) were used in this study. The performance of SCC was evaluated primarily at fresh state. Produced concrete sample were tested for compressive strength after standard curing period. The study gives an insight on suitability of particular type of chemical admixtures for SCC production.

Keywords: Self-compacting concrete (SCC), Fly ash, Viscosity, HRWR.

1. INTRODUCTION

Self – compacting concrete (SCC) has the ability to flow under its own weight and can achieve full compaction without any mechanical vibration. The concrete can be self-compacted and achieve mechanical and durability properties comparable to that of traditionally compacted concrete (BIBM *et al*., 2005; Bouzouba and Lachmi, 2001). The concept was developed in Japan during 1980s and then it has gained interest in Europe during 1990s. Benefits of SCC includes faster construction, resistance to segregation, homogeneity and minimum voids, uniform strength and thereby improved finish and durability. Not only the construction environment the workers also benefits from elimination of noise and vibration which is produced from mechanical compaction process (BIBM *et al*., 2005).

On the other hand construction industry requires achieving sustainability by reducing production of cement and use of other virgin construction materials. The world is therefore in search of alternative cementitious materials. Appropriate use of fly ash in construction can improve performance of produced concrete both in fresh and hardened state (Islam, 2012). In Bangladesh, Barapukuria Power Plant is producing more than 65 thousand tons of fly ash per year (Islam *et al*., 2011). This work aimed to explore possibilities of using fly ash in production of Self-Compacting Concrete.

2. SCC DESIGN CONSIDERATIONS

Basic requirement for SCC is to achieve excellent deformability. Additionally, it should have a low risk of blockage, and good stability which would ensure high formwork filling capacity. Easy flow ability in highly congested areas makes SCC to be a suitable material for the construction of structural members with congested of steel reinforcement.

This is a special type of high performance concrete. According to Okamura and Ouchi (2003) SCC should not only be self-compactable at fresh state, it should free from initial defects at early age and protect the reinforcement against external factors upon hardening. Therefore, this sometimes called self-compacting high performance concrete. For SCC, the concrete should have high deformability of paste/mortar at the same time the segregation between coarse aggregate and mortar should be minimum. To achieve self-compatibility, basic mix design for conventional concrete has been modified. This is graphically shown in Figure 1. The collision and contact between coarse aggregates (intense energy consumption) can increase internal stress and thereby results blockage at obstacles. It is recommended that the gravel content should not exceed 50% of solid volume (Okamura and Ouchi, 2003). EFNARC (2005) allows somewhat larger values up to 60%.

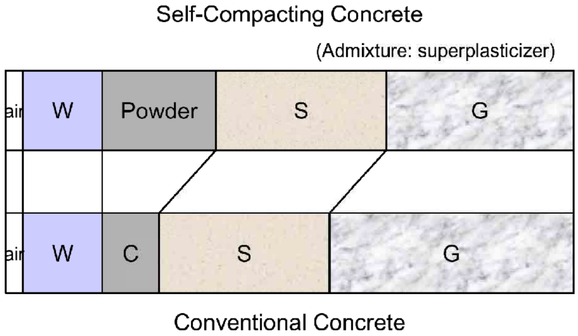

Figure 22: Comparision of self-compacting concrete with conventional concrete (Okamura and Ouchi, 2003)

Sand content should be limited to 40% of mortar volume as per Okamura and Ouchi (2003) while EFNARC (2005) allows up to 50%. Paste viscosity needs to be higher to avoid blockage. Therefore, to achieve higher deformability, a higher dose of superplasticizer would require. The w/c ratio should be lower to achieve moderate viscosity. The degree of packaging of sand in SCC needs to be around 60% to keep share deformability of concrete at minimum level. Lower water-powder ratio gives low viscosity which results segregation control (Okamura and Ouchi, 2003).

3. TESTING SCC

To check achievement of self-compatibility, several test methods are developed by various organizations. These purposes of these tests include, a) checking whether the concrete is self-compactable for structures; b) adjustment of mix proportions if self-compatibility is not achieved; and c) material characterization. The former can check with U-flow or box test. Taisei group of Japan has been proposed the U-flow test in 1993 (Hayakawa, 1993). If concrete can pass through the specified obstacle and fill higher than 300 mm the concrete can be considered as self-compacting concrete. Where the concrete possess higher possibility of segregation, the box test can be more effective. In case the concrete does not pass the test, the mix design needs to modify. Traditional slump-flow and funnel tests were also used by researchers to test deformability and viscosity of the respectively. EN 206-9 (CEN, 2010) mentioned that V-funnel test which indicates viscosity of the mix is suitable for maximum aggregate size larger than 22.4 mm. The same document also refers L-Box test for evaluating passing ability of the matrix. The above two test devices were constructed in the central workshop of CUET for testing SCC. Figure 2 shows the fabricated instruments.

(a)

(b)

Figure 23: SCC fresh properties testing devices. (a) L-box and (b) V-funnel (Arif & Kabir, 2013)

4. MATERIALS

4.1 Aggregates

Aggregates are obtained from a ready-mix concrete company. Care has been taken to match ASTM ¾″ size for stone chips. Larger sized aggregates were removed manually. Large grain sand obtained from Sylhet was used as fine aggregates. Key properties of fine and coarse aggregates are given in Table 1.

Table 24. Physical properties of aggregates

Properties	Unit	Aggregate Type	Result	Ref. Method
Absorption capacity	%	Fine	1.57	BS 812:Part 107
		Coarse	0.95	
Specific Gravity	--	Fine	2.64	ASTM C 128-03
		Coarse	2.70	
Flakiness Index	%	Coarse	16.8	BS 812: Part 1
Fineness Modulus	--	Fine	2.94	ASTM 136-03

4.2 Fly Ash and Cement

Ordinary Portland Cement (OPC) was used for all tests. Fly ash from Barapukuria Power Plant (only coal based power plant in Bangladesh) was obtained. The Barapukuria Coal Mine was discovered in 1985 by the Geological Survey of Bangladesh (GSB). At this moment the mine is being operated by Barapukuria Coal Mine Company Limited (BCMCL) and the authority targeted 1.1 million metric ton of coal to mine during September 1013 to August 2014. The coal contains 27.71 MJ/kg heating value which is ideal for power generation without preparation or treatment of the coal. According to BCMCL (2013) the coal produces approximately 12% fly ash after power generation. The current year's production therefore would produce 132 thousand metric tons of fly ash which requires attention for further processing or reuse. The route of using this material in concrete can save the environment and can also produce better concrete (Islam *et al*., 2011). Chemical and physical properties (using XRF) of OPC and fly ash used in the project are given in Table 2.

Table 25: Properties of OPC and Fly Ash

Constituents	Composition	OPC	Fly Ash
Calcium Oxide	CaO	65.2	0.7
Silicon Di-Oxide	SiO_2	20.8	50.5
Aluminum Oxide	Al_2O_3	5.2	32.0
Ferric Oxide	Fe_2O_3	3.2	2.8
Magnesium Oxide	MgO	1.2	0.6
Sulfur Tri-Oxide	SO_3	2.2	1.0
Sodium Oxide	Na_2O	0.1	0.30
Loss on Ignition	--	1.7	3.8
Insoluble Residue	--	0.6	--
Blaine Fineness, cm^2/gm	---	4000	3800

4.3 Superplasticizer

This research worked with two commercially available superplasticizers in Bangladesh. Table 3 gives key physical properties of the superplasticizers. SP R is specially designed for keeping the slump for longer while SP G supposed to give high early strength by reducing water demand. The recommended dose for the former is higher than that of the later which indicating the latter would be better in superplasticizing effect.

Table 26. Physical Properties of Aggregates

Superplasticizer	SP R	SP G
Specific Gravity	1.25	1.09
Origin	Synthetic polymers	Polycarboxylic ether polymer
Aspect	Dark brown free flowing liquid	Light brown liquid
Recommended dose	600 – 1800 ml per 100 kg of cementitious material	500 – 1200 ml per 100 kg of cementitious material
Application	Water reduction & retardation	Water reduction & high early strengths

5. METHODOLOGY

5.1 Mix Design

To formulate the mix design, initially traditional mix design recommended by ACI 211.1 (2009) was reviewed. The guideline by Okamura and Ouchi (2003), EFNARC (2005) were also taken into consideration. Mix proportions used by various researches were also reviewed and finally for trial mix a total of 18 mix proportions adopted. The mix designs used for trial mix of SCC in this research are given in Table 4. Initially, Mix ID M1-M13 was carried out using SP R and the rest with SP G. Fly ash was used for last 3 mixes at a level of 10, 20, and 30% of total cementitious materials for M16, M17 and M18, respectively. The first 3 mixes were with water/powder (w/p) ratio around 0.42 and then it was changed to around 0.32 for later mixes. Sand content were kept between 56-58% of mortar volume as per EFNARC (2005). The coarse aggregate content were ranged between 49 – 51% of total aggregate content.

Table 27. Mix Design of SCC Concrete

Mix ID	Quantity (kg/m^3)						Sand*, %	Gravel**, %
	Water	Cement	Fly Ash	Sand	Gravel	Superplasticizer		
M1	185	465	0	865	840	9.3	57	49
M2	190	465	0	875	840	10.0	57	49
M3	205	465	0	865	840	10.0	56	49
M4	155	470	0	870	840	11.8	58	49
M5	155	470	0	870	840	9.4	58	49
M6	155	480	0	870	835	7.2	58	49
M7	155	480	0	870	840	6.7	58	49
M8	150	485	0	870	840	5.8	58	49
M9	150	490	0	870	840	6.4	58	49
M10	155	490	0	870	835	7.4	57	49
M11	155	490	0	870	840	7.4	57	49
M12	155	490	0	870	840	22.1	57	49
M13	155	490	0	860	850	16.8	57	50
M14	155	490	0	855	850	3.7	57	50
M15	155	490	0	850	870	4.2	57	51
M16	155	440	50	830	840	4.4	56	50
M17	155	390	100	825	835	4.4	56	50
M18	155	345	145	825	835	4.4	56	50

** Sand content of total mortar content, Gravel content of total aggregate content.*

5.2 Mixing and Testing of Concrete

A drum type concrete mixture machine has been employed to mix the concrete of 30 litre batch. Aggregates were made to saturated and surface dry condition so that no water adjustments were required. Superplasticizers were dispensed into the mixing water before pouring into the mixture machine. As with superplasticizer fly ash was blended with cement thoroughly before placing inside the mixture machine. Initially, sand and gravel is placed inside the machine and mixed thoroughly. After that cement is places and mixed for 30 second. Full quantity of weighted water with superplasticizer was employed inside the machine. After mixing all the ingredients thoroughly the mix was taken out of the machine and slump flow test was conducted as soon as possible.

As indicated in the literature the slump flow needs to be in the range of 650-800 mm to consider the concrete as SCC. The quantity of ingredients in mix proportions were played around in this research to achieve this targeted value. The target was not achieved with SP R until mix M11, keeping the superplasticizer dose within recommended dose range. Considerably high dose of SP R was used for mix M13 and M13. This gave considerable higher slump flow however, due to excessive dose, the concrete took very long time to set. After 3 days of casting, this did not set and looked like a soft cake as shown in Fig 2(a). A major property of SP R is extending the setting time. Though the mix gave an initial flow close to the fresh property of SCC, its hardened property is not suitable for the desired purpose. Mix M14 gave maximum of 455 mm slump flow. Figure 2(b) shows slump flow test of fresh SCC. However, having an early setting property, SCC with SP G was set rapidly (within 15 minutes) to measure the slump flow.

(a) (b)

Figure 2: (a) Effect of admixture for high dosing, and (b) Slump Flow diameter

The L-box test was carried out for few samples but they were not promising to present. The research, therefore, found both SP R and SP G are inappropriate to use for SCC. Admixtures based on polycarboxylic ether polymer or synthetic polymer is not suitable for SCC. Though these gave an initial flow, these set rapidly. As the superplasticizers were not appropriate for SCC, no effect of fly ash on fresh properties was noted. EFNARC (2006) suggests using Viscosity Modifying Admixtures (VMA) to achieve self-compactability.

5.3 Strength Results

Compressive strength of 100 mm cubic concrete specimens was tested at 7 and 28 days for M14, M16, M17 and M18 mixes. M14 mix is control with no fly ash while M16, M17 and M18 contained 10, 20 and 30% fly ash. Cubic moulds were filled with fresh concrete after testing fresh property. The concrete was compacted in two layers and finished the surface. These was de-moulded after 24 hours of mixing and then put under water until the test age of 7 and 28 days.

The compressive strength was ranged between 27.2 – 28.8 MPa and 42.2 – 48.3 MPa at 7 and 28 days, respectively. Figure 3 shows trends in compressive strength development with fly ash level. As found elsewhere and expected, the strength of fly ash samples were found to be slightly lower than that of control OPC concrete (Islam, 2012).

6. CONCLUSIONS

The current research explored possibility of producing SCC with fly ash from local source. In this regard V-funnel and L-box were constructed at the central workshop of CUET to test fresh properties of the concrete. Using two different types of superplasticizers, it was found that dose higher than that recommended by the manufacturer of retarder and water reducer can increase flowability of the concrete. However, this affects the setting of produced concrete, considerably. Using early setting and water reducing admixture also gave higher flow than conventional concrete, however, this suffered with rapid setting. Therefore, further study with viscosity modifying admixture is recommended from this study. The strength of produced SCC with fly ash at 28 days was found to be comparable with that of control concrete with OPC.

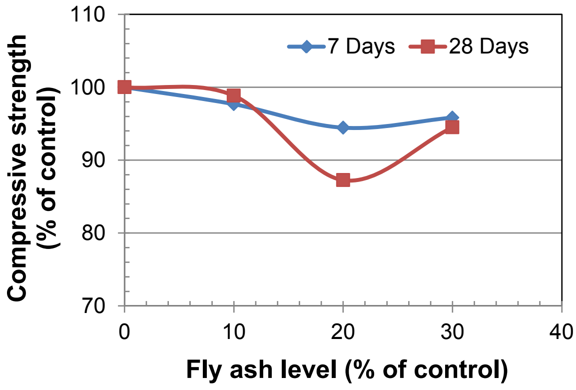

Figure 3: Strength of SCC samples using various fly ash content

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REFERENCES

ACI 211 (2009). "Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete," ACI Committee 211. ACI Manual of Concrete Practice, Part 1, American Concrete Institute, 38800 Country Club Dr., Farmington Hills, MI, 48331-3439, USA.

Arif, S. and Kabir, F. (2013). Self Compacting Concrete by using Barapukuria Fly Ash". Undergraduate Thesis, Department of Civil Engineering, Chittagong University of Engineering & Technology (CUET), Chittagong 4349, Bangladesh.

BIBM, CEMBUREAU, ERMCO, EFCA and EFNARC (2005). "The European Guidelines for Self Compacting Concrete: Specification, Production and Use", www.ermco.eu/documents/ermco-documents/scc_guidelines_may_2005_final.pdf.

Barapukuria Coal Mine Company Limited (BCMCL) (2013). http://www.bcmcl.org.bd/

Bouzouba, N. and Lachmi, M. (2001). "Self-compacting concrete incorporating high volumes of class F fly ash, preliminary results", Cement and Concrete research, Vol. 31(3), 413-420.

Campion, J. M. and Jost, P. (2000). Self-compacting: Expanding the possibility of concrete design and placement. Concrete International, Vol 22(4), 31-34.

EFNARC (2006). "Guidelines for Viscosity Modifying Admixtures For Concrete". European Federation of Concrete Admixture Association.

European Committee for Standardization (CEN) (2010). "Concrete, Part 9: Additional Rules for Self-Compacting Concrete (SCC)", EN 206-9, Avenue Marnix 17, B-1000, Brussels, Belgium.

Hashimoto, C. (1989). Study on visualization technique for blocking fresh concrete flowing in pipe, Concrete Library International, JSCE (12), 139-153.

Hayakawa, M., Matsuoka, Y. and Shindoh, T. (1993). "Develpment and aplication of super workable concrete". *RILEM Internationals Workshop on Special Concretes: Workability and Mixing.*

Islam, G. M. S. (2012). "Evaluating Reactivity and Sorptivity of Fly Ash for Use in Concrete Construction", PhD Thesis, August 2012, University of Dundee, United Kingdom.

Islam, G. M. S., Islam, M. M., Akter, A. and Islam, M. S. (2011). "Green Construction Materials – Bangladesh Perspective". In International Conference on Mechanical Engineering and Renewable Energy (ICMERE2011), 22-24 December 2011, Chittagong, Bangladesh.

Okamura, H. and Ouchi, M. (2003). "Self-compacting concrete". Journal of Advanced Concrete Technology, JCI, Vol. 1(1), 5-15.

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APPLICATIONS OF FIBER REINFORCED POLYMER COMPOSITES (FRP) IN CIVIL ENGINEERING

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ABSTRACT

There is a growing concern with worldwide deterioration of traditional materials such as concrete, steel, and timber. Recently, attention has shifted to the use of fiber reinforced polymer composites (FRPs) as alternative materials. As FRPs are non-corrosive, high strength and modulus values compared to their density, light weight, acceptable deformability, tailored design and excellent formability enable the fabrication of new elements and the structural rehabilitation of the existing parts made of traditional materials. Furthermore, the resistance of FRP materials to corrosion means that they can be used to replace steel and reinforced concrete in situations when they would be exposed to corrosion. FRP therefore has wide application prospects in civil engineering ranging from reinforcing rods and tendons, wraps for seismic retrofit of columns and externally bonded reinforcement for strengthening of walls, beams, and slabs, to all-composite bridge decks, and even hybrid and all-composite structural systems. This paper is a review of the application of FRPs in civil engineering. Firstly, the paper will elucidate the basic information about FRP composites, including the definition, description of the components such as fibers and matrices. Then it pointed some fabrication processes, mechanical properties. Finally, it will focus on the application of FRP in civil engineering.

Keywords: FRP, composite, reinforcement, matrix, rehabilitation.

1. INTRODUCTION

Fiber-reinforced polymer composite materials (FRP) have hitherto been utilized predominantly in the aerospace and military industries, but for the last three decades there has been a growing awareness amongst civil engineers of the importance of the unique mechanical and in-service properties of these materials together with their customized fabrication techniques. In fact, this class of materials presents an immense potential for use in Civil Engineering, both for rehabilitation of existing structures and for the construction of new facilities (Lopez-Anido *et al.* 2000).

Polymer composites are multi-phase materials produced by combining polymer matrix with fillers and reinforcing fibers to produce a bulk material with properties better than those of the individual base materials. The matrix can be thermoplastics (Polypropylene, polyethylene, polystyrene, PVC (polyvinyl chloride) etc.) or thermosetting (Polyester, vinyl ester, epoxy resins etc.). Fillers are often used to bulk to the material, reduce cost, lower bulk density or to produce aesthetic features. Fibers are used to reinforce the polymer and improve mechanical properties such as stiffness and strength. High strength fibers of glass, aramid and carbon are used as the primary means of carrying load, while the polymer matrix protects the fibers and binds them into a cohesive structural unit. These are commonly called fiber-reinforced polymer composite materials (FRPs).

Advanced composite materials have found expanded use in aerospace, marine and automobile industries during the past few decades (1960 onwards) due to their good engineering properties such as high specific strength and stiffness, lower density, high fatigue endurance, high damping and low thermal coefficient (in fiber direction), etc. Recently, civil engineers and the construction industry have begun to realize potential of composites as strengthening material for many problems associated with the deterioration of infrastructures. Over the last decade, an increase in the application of FRPs has been seen in construction industry because of their good engineering properties. Further, these are being considered as a replacement to the conventional steel in reinforced concrete structures due to continuing drop in the cost of FRP materials. Various aspects of FRPC materials including guidelines for selection of polymer adhesives for concrete have been highlighted by ACI Committee-503 (1992) and Uomoto et al. (2002). Issues related to selection of materials have also been discussed by Karbhari (2001). Einde et al. (2003) and Bank et al. (2003) have presented a summary of applications of FRP material in civil engineering whereas general design guidelines for FRP application can be found in Bakht et al. (2000), ACI Committee 440 (2002) and Nanni (2003).
Use of FRP sheets for strengthening and rehabilitation of concrete structures has attracted considerable interest *(Nanni et al. 1993, Mufti at al. 2002, Hollaway et al. 2003, Mufti et al. 2003)*. First applications of composites

were in the form of rebars and structural shapes. Later, FRP laminates were used for strengthening of concrete bridge girders by bonding them to the tension face of girder (Meier *et al.* 1992) as well as for retrofitting of concrete columns (Saadatmanesh 1994).

FRPs are available in the form of rods, grids, sheets and winding strands. Review of literature up to 1996 can be found in ACI Committee 440 (1996). Another general review on class of materials including FRPs used in civil construction was presented by Bakis et al. (2002). They divided the whole review into structural shapes, internal reinforcement, externally bonded reinforcement, bridge, standards and codes. A review on shear strengthening of RC beams with FRPCs was done Deniaud and Cheng (2001), Bousselham and Chaallal (2004). Review related to the bond-slip model for FRP sheet/plate bonded to concrete have presented recently by Lu et al. (2005) and review for upgrading of beam-column joints with FRP can be found in Engindeniz et al. (2005). A large volume of literature now exists on applications of FRPs in construction industry.

2. FIBER REINFORCED POLYMER COMPOSITES (FRP)

For An FRP is a specific type of two-component composite material consisting of high strength fibers embedded in a polymer matrix. The mechanical and physical properties are clearly controlled by their constituent properties and by the micro-structural configuration. While the fibers are mainly responsible for strength and stiffness properties, the polymeric matrix contributes to load transfer and provides environmental protection. In addition, fillers are used to reduce the cost and sometimes to improve performance, imparting benefits as shrinkage control, surface smoothness and crack resistance. Additives and modifiers ingredients can expand the usefulness of the polymeric matrix, enhance their processability or extend composite durability.

The reinforcing of a low modulus polymeric matrix with high strength and modulus fibers utilizes the viscoelastic displacement of the matrix under stress to transfer the load to the fiber; this result in a high strength, high modulus composite material. The aim of the combination is to produce a two phase material in which the primary phase, that determines stiffness, is in the form of fibers and is well disperse and bonded and protected by a weak secondary phase, the polymeric matrix (Hollaway and Head 2001).

2.1 Reinforcing Fibers

The fibers provide the strength and stiffness of an FRP. Because the fibers used in most structural FRP applications are continuous and are oriented in specified directions, FRPs are orthotropic, and they are much stronger and stiffer in the fiber direction(s). According to Halliwell (2000), the functional requirements of fibers in a composite are:

i. High modulus of elasticity to give stiffness
ii. High ultimate strength
iii. Low variation of strength between individual fibers
iv. Stability during handling
v. Uniform diameter

Generally fiber can be used in different ways, with the performance changing for each (Cripps 2002):

- The highest performance in terms of strength and stiffness in one direction comes from unidirectional composites, when fibers are parallel and give their maximum possible performance in this single direction,
- By arranging the fibers in a weave or mat, strength can be gained in more directions, although the limit strength is reduced,
- By chopping the fibers into short lengths and arranging them randomly, equal strength is achieved in all directions. This is generally the cheapest technique, used for the least structurally demanding cases.

Many different types of fibers are available for use, and all have their respective advantages and disadvantages. In civil engineering applications, the three most commonly used fiber types are glass, carbon (graphite), and to a lesser extent, aramid (Kevlar). The suitability of the various fibers for specific applications depends on a number of factors including the required strength, the stiffness, durability considerations, cost constraints, and the availability of component materials.

2.1.1 Glass fibers

Glass fibers are commonly produced by a process called direct melt, wherein fibers with a diameter of 3 to 25 microns are formed by rapid and continuous drawing from a glass melt. Glass fibers are used for the majority of composite application because they are cheaper than the others. There are different forms known by names like E-glass (the most frequent used), S-glass (is a stringer and stiffer fiber with a greater corrosion resistance), R-glass (is a higher tensile strength and modulus and greater resistance to fatigue and aging) and AR-glass (an alkali-resistant glass used to reinforced concrete). The main characteristics of glass fibers are their high tensile strengths and moderate elastic modulus. Glass fibers are, also, excellent thermal and electrical insulators. Glass fibers are particularly sensitive to moisture, especially in the presence of salts and elevated alkalinity, and need to be well protected by the resin systems used in the FRP. Glass fibers are also susceptible to creep rupture and lose strength under sustained stresses.

2.1.2 Carbon fibers

Carbon fibers are produced by a process called controlled pyrolysis, wherein one of three potential precursor fibers is subjected to a complex series of heat treatments (stabilization, carbonization, graphitization, and surface treatment) to produce carbon filaments with diameters in the range of 5-8 microns. The resulting fibers can have properties that vary widely, and so several classes of carbon fibers are available, differentiated based on their elastic moduli: Standard: 250-300 GPa, Intermediate: 300-350 GPa, High: 350-550GPa, Ultra-High: 550-1000 GPa. Although considerably more expensive than glass fibers, carbon fibers are beginning to see widespread use in structural engineering applications such as pre-stressing tendons for concrete and structural FRP wraps for repair and strengthening of reinforced concrete beams, columns, and slabs. Their steadily increasing use can be attributed to their steadily decreasing cost, their high elastic moduli and available strengths, their low density (low weight), and their outstanding resistance to thermal, chemical, and environment effects, they do not absorb moisture. Carbon fibers are an ideal choice for structures which are weight and/or deflection sensitive.

2.1.3 Aramid fibers

Aramid fibers are manufactured from a synthetic compound called aromatic polyamide in a process called extrusion and spinning. In this fiber, molecular chains are aligned and made rigid by means of aromatic rings linked by hydrogen bridges. Their main characteristics are high strength, impact resistance due to their energy absorbing capacity properties, moderate modulus and low density. In addition, FRPs manufactured from aramid fibers have low compressive and shear strengths as a consequence of the unique anisotropic properties of the fibers. The fibers, themselves, are susceptible to degradation from ultraviolet light and moisture but exhibit resistance to acids and alkalis.

2.1.4 Basalt fibers

Basalt fibers are materials obtained by melting crushed volcanic lava deposits. Basalt fibers have better physical and mechanical properties than glass fibers, but are significantly cheaper than carbon fibers. Their main advantages are fire resistance, significant capability of acoustic insulation and immunity to chemical environments.

Table 1 illustrates typical properties of the different types of fibers and steel, showing their strength, modulus and density.

Table 1: Range of properties for fibers for FRP composites

Fiber type	Density (Kg/m^3)	Tensile strength (GPa)	Elastic Modulus (GPa)
Glass	2.46 – 2.58	2.4 – 3.5	72 – 87
Carbon	1.74 – 2.20	2.1 – 5.5	200 – 500
Aramid	1.39 – 1.47	3.1 – 3.6	58 – 130
Basalt	2.65 – 2.80	4.2 – 4.8	89 – 110
Steel	7.85	480 – 700	200

Other fibers that are now in the development phase for use if FRP products for structural engineering include ultrahigh-molecular-weight polyethylene fibers and polyvinyl alcohol fibers. Natural fibers, such as sisal, flax and bamboo, have been used only in experimental applications to produce FRP products. However, it is expected that they will become more important in the construction industry due to their sustainability and recyclability (Bank 2006).

2.2 Matrix

The matrix is the binder of the FRP and plays many important roles. Some of the more critical functions played by the matrix are:

- To bind the fibers together
- To protect the fibers from abrasion and environmental degradation
- To separate and disperse fibers within the composite

- To transfer force between the individual fibers and
- To be chemically and thermally compatible with the fibers.

According to Hollaway and Head (Elsevier 2001), the requirements for a good FRP matrix are the following:

i. Wet out the fiber and cure satisfactory in the required conditions
ii. Bind together the fibers and protect their surface from abrasion and environmental ageing
iii. Disperse the fibers as separate them in order to avoid any catastrophic propagation of cracks
iv. Transfer stresses to the fibers efficiently
v. Be chemically and thermally compatible with fibers
vi. Have appropriate fire resistance and limit smoke propagation
vii. Provide good aesthetic finish (color and surface).

There are several different polymer matrices which can be utilized in FRP composites, but in construction industry only a relatively small number are actually used. According to their nature, there are two major types of polymers, which determine the methods of manufacturing and the properties of the composite: (i) thermoplastic and (ii) thermosetting. The first FRP were all based on thermosetting polymers and, besides the fact that thermoplastic have seen rapid growth in recent years, thermosetting is yet the most used in Civil Engineering applications (ACI 440R 1996).

2.2.1 Thermoplastic Matrix

Thermoplastics are polymers composed of long-chain molecules that are held together by relatively weak Van der Waals forces, but that have extremely strong bonds within individual molecules. These polymers can be amorphous, which implies a random structure with a high concentration of entanglement, or crystalline, with a high degree of molecular order (Cowie 1991). In these materials, the molecules are free to slide over one another at elevated temperatures, and so thermoplastics can be repeatedly softened and hardened by heating and cooling without significantly changing their molecular structure. The semi-crystalline polypropylene and nylon are especially popular as matrices.

2.2.2 Thermosetting Matrix

Thermosetting polymers are also long-chain molecules built from monomers, but for these materials the molecular chains are cross-linked through primary chemical bonds. Thus, thermosets cannot be reversibly softened and will deteriorate irreversibly at elevated temperatures. These are usually made from liquid or semi-solid precursors which harden irreversibly; this chemical reaction is known as cure and on completion, the liquid resin is converted to a hard solid by chemical cross-linking which produces a tightly three-dimensional network of polymer chains. Almost exclusively, thermosets are currently used in structural engineering applications. These polymers generally have good thermal stability at service temperatures, good chemical resistance, and display low creep and relaxation properties in comparison with most thermoplastics. However, because it is difficult to reversibly soften thermosets, FRP components made from thermosets matrices must be bent or formed during the manufacturing process. This may become a problem in some specific applications. For example, FRP reinforcing bars for concrete that incorporate thermosetting polymer resins cannot be bent on site, and research is currently underway to develop satisfactory thermoplastic matrices for these specialized applications.

Three specific types of thermosetting resins are commonly used in the manufacture of infrastructure composites: polyester resin, epoxy resin and vinylesters resin.

2.2.2.1 Polyester Resin

Polyesters are the most widely used polymers in the manufacture of FRP components for infrastructure applications due to their relatively low cost and ease of processing (these resins cure at ambient temperatures). Numerous specific types of polyesters are available for use, with varying degrees of thermal and chemical stability, moisture absorption, and shrinkage during curing.

2.2.2.2 Epoxy Resin

Epoxies are often used in wet lay-up applications of FRP plates and sheets because of their ability to cure well at room temperature and owing to their outstanding adhesion (bonding) characteristics. Epoxies have high strength, good dimensional stability, relatively good high-temperature properties, strong resistance to chemicals (except acids), and superior toughness. Epoxies, however, cost significant more than polyesters or vinylesters

2.2.2.3 Vinylester Resin

Vinylesters have similar mechanical and in-service properties to those of the epoxy resins and equivalent processing techniques to those of the unsaturated polyesters. Vinylesters are resistant to strong acids and alkalis, which is one reason that they are commonly used in the manufacture of FRP reinforcing bars for concrete (the environment inside concrete is highly alkaline). They also offer reduced moisture absorption and shrinkage as compared with polyesters. Vinylesters cost slightly more than polyesters.

2.3 Fabrication of FRP Composites

There is a wide variety of techniques by which FRP composites can be fabricated, although there are differences between the techniques available for thermosetting and thermoplastic, due to their intrinsic different properties. Table 2 presents the commonly used process for fabrication of FRP composites applied in Civil Engineering, heir principles and typical applications (Cripps 2002).

Table 2: Fabrication processes of FRP composites

Pultrusion	Tightly packed tows of fibers, impregnated with polymer, are pulled through a shaped heated die to form aligned, continuous sections geometry. Solid and hollow profile section may be produced with a high fiber content and high degree of fiber alignment. Off-axis fibers may also be introduced, if required. Pultruded shapes and concrete reinforcing bars and tendons; I beams and other sections.
Filament Winding	The process involves winding fibers over a mandrel which rotates while a moving carriage laying down the reinforcement in the desired pattern. The orientation of the fibers can also be carefully controlled so that successive layers are plies or oriented differently from the previous layer. Cylindrically symmetric structures such as hollow and vessels. The process of wrapping in retrofit strengthening is an adaptation of the process.
Compression and Transfer Moulding	Compression moulding of thermosetting moulding compounds in dough with chopped glass fibers (DMC) or sheets with longer fibers (SMC). Simple or complex decorative panels.
Matched-die Moulding and Autoclave	Large panels and relatively complex open structural shapes are constructed by hot-pressing shets of pre-impregnated fibers or cloths between flat or shaped platens, or by pressure autoclaving to consolidate a stack of prepreg sheets against a heated, shaped die. Composite reinforced with chopped-strand mat or continuous-filament mat reinforcements may also be press-laminated. Laminates and retrofit strengthening sheets.
Continuous Sheet Production	Chopped strand mat or chopped strands are impregnated with resin and sandwiched between two layers of film on a moving belt. The sandwich passes through guides tha form the corrugated or other desired profile. Corrugated plates.
Resin Transfer Moulding and Vacuum-assisted resin transfer moulding	Pre-catalysed resin is pumped under low pressure into a fiber perform, which is contained in a closed and often heated die. The preform may be made of any kind of reinforcement, but usually consists of woven cloths or continuous-fibers mats. Structural components with varying shapes and degrees of anisotropy/orthotropy, e.g. cladding and roofing panels, shell structures and bridge decks.
Contact moulding by hand lay-up or spray-up	Open mould methods, where fiber continuous strand mat and/or other fabrics such as woven roving are placed manually in the mould and each ply is impregnated with brushes and rollers. The product must also be built by spraying through a gun which simultaneously delivers short fiber and pre-catalysed resin. Fabrication one-off structures, small number of large components.

2.4 Mechanical Properties of FRP Composites

The mechanical properties of an FRP depend on a number of factors including:

- The relative proportions of fiber and matrix
- The mechanical properties of the constituent materials (fiber, matrix, and any additives)
- The orientation of the fiber within the matrix, and
- The method of manufacture.

The Young's modulus and tensile strength of composites are lower than that of fibers alone. The volume fraction of fibers normally ranges between 50-65%. Thus each FRP composite has its own typical mechanical characteristics which make it suitable for a given structural application. Glass fibers are considerably cheaper than carbon fibers but some forms of this fiber tend to be very sensitive to the alkaline environments of concrete. Glass fibers also have a lower elastic modulus than carbon fibers. A comparison of mechanical properties of FRP with steel is provided in Table 3 (Mufti 1991).

Table 3: Typical comparative properties of FRP and steel.

Material	Tensile Strength (MPa)	Modulus of Elasticity (MPa)
Glass-Epoxy	1050	55000
Carbon-Epoxy	1500	180000
Aramid-Epoxy	1400	76000
Steel	400-1000	200000

Regardless of the type of fibers employed, FRP materials have similar stress-strain behavior: linear elastic up to final brittle rupture when subject to tension, which means that they do not possess the ductility that steels have, and their brittleness limit the ductile behavior of RC members strengthened with FRP composites. Nevertheless, when used to provide confinement for concrete, these materials can greatly enhance the strength and ductility of columns.

Polymeric resins are used both as the matrix for the FRP and as the bonding adhesive between the FRP and the concrete. The latter function is of particular concern here, as weak adhesives can cause interfacial failures. Epoxy resins are generally used when the bonding function of resins is of crucial importance, as in the flexural and shear strengthening of beams. Present-day epoxy resins are so strong that interfacial failures generally occur in the concrete, particularly as the concrete in the structure to be strengthened is comparatively weak.

Some commonly available FRPs used in concrete reinforcing applications, and their respective properties, are listed in Tables 4 and 5. Table 6 provides a comparison between various types of FRPs and conventional reinforcing materials for concrete. From this data it is evident that both glass and aramid FRPs have moduli that are considerably less than steel in the pre-yield zone, but that carbon FRPs have moduli that are comparable to, or even higher than, steel in some cases. Also evident from the data is the fact that FRPs have ultimate strengths that can be many times greater than steel.

Table 4: Selected properties of typical currently available FRP reinforcing products

Reinforcement Type	Designation	Diameter (mm)	Area (mm²)	Tensile Strength (MPa)	Elastic Modulus (GPa)
Deformed Steel	#10	11.3	100	400*	200
V-ROD CFRP Rod	3/8	9.5	71	1431	120
V-ROD GFRP Rod	3/8	9.5	71	765	43
NEFMAC GFRP Grid	G 10	N/A	79	600	30
NEFMAC CFRP Grid	C 16	N/A	100	1200	100
NEFMAC AFRP Grid	A 16	N/A	92	1300	54
LEADLINE™ CFRP Rod	Round	12	13	2255	147

*specified yield strength

Table 5: Selected properties of typical currently available FRP strengthening systems*

FRP System	Fiber Type	Weight (g/m^2)	Thickness (mm)	Tensile Strength (MPa)	Tensile Elastic Modulus (GPa)	Strain at Failure (%)
Fyfe Co. LLC (www.fyfeco.com)						
Tyfo SHE-51	Glass	930	1.3	575	26.1	2.2
Tyfo SCH-35	Carbon	-	0.89	991	78.6	1.3
Mitsubishi (www.mitsubishichemical.com)						
Replark	Carbon	200	0.11	3400	230	1.5
Replark 30	Carbon	300	0.17	3400	230	1.5
Replark MM	Carbon	-	0.17	2900	390	0.7
Replark HM	Carbon	200	0.14	1900	640	0.3
Sika (www.sika.com)						
Hex 100G	Glass	913	1.0	600	26.1	2.2
Hex 103C	Carbon	618	1.0	960	73.1	1.3
CarboDur S	Carbon	2240	1.2-1.4	2800	165	1.7
CarboDur M	Carbon	2240	1.2	2400	210	1.2
CarboDur H	Carbon	2240	1.2	1300	300	0.5
Degussa Building Systems (www.wabocorp.com)						
MBrace EG 900	Glass	900	0.35	1517	72.4	2.1
MBrace CF 530	Carbon	300	0.17	3500	373	0.94
MBrace AK 60	Aramid	600	0.28	2000	120	1.6

*Additional information can be obtained from the specific FRP manufacturers

Table 6: Comparison of typical approximate properties for reinforcing materials for concrete**

Property	Steel Rebar	Steel Tendon	GFRP Rebar	CFRP Tendon	AFRP Tendon
Tensile Strength (MPa)	483-690	1379-1862	517-1207	1200-2410	1200-2068
Yield Strength (MPa)	276-414	1034-1396	N/A	N/A	N/A
Tensile Elastic Modulus (GPa)	200	186-200	30-55	147-165	50-74
Ultimate Elongation (%)	>10	>4	2-4.5	1-1.5	2-2.6
Compressive Strength (MPa)	276-414	N/A	310-482	N/A	N/A
CTE* ($10^{-6}/^{o}C$)	11.7	11.7	9.9	0	-1 – 0.5
Specific Gravity	7.9	7.9	1.5-2.0	1.5-1.6	1.25

**FRP materials are continually being developed with better properties. The properties given are circa 2000.
*coefficient of thermal expansion (CTE)materials.

2.5 Environmental Durability

FRP materials are increasingly being used in civil engineering applications such as reinforcing rods and tendons, wraps for seismic retrofit of columns, externally bonded reinforcement, composite bridge decks, and even hybrid and all composite structural systems. Since FRP are still relatively unknown to the infrastructure system planner, there are heightened concerns related to the overall durability of these materials, especially as related to their capacity for sustained performance under harsh and changing environmental conditions under load (Karbhari 2007).

Although FRP have been successfully used in the industrial, automotive, marine and aerospace sectors, there are critical differences in loading, environment and even the types of materials and processes used in these applications. Several evidences provides substantial reason to believe that if appropriately designed and fabricated, these materials can grant longer lifetimes and lower maintenance costs than equivalent structures fabricated from conventional materials (Karbhari 2007).

FRP materials used in civil infrastructure are exposed to a variety of environmental that may act individually or may be synergistic in nature. According a recent study undertaken to identify critical gaps in durability of composites to be used in civil engineering applications (Karbhari *et al.* 2003), seven factors were distinguish, namely:

i. Moisture/solution
ii. Alkali
iii. Thermal, including cycling and freeze-thaw
iv. Creep and relaxation
v. Fatigue
vi. Ultraviolet radiation
vii. Fire.

3. APPLICATIONS

Over the last decade there has been significant growth in the use of FRP composites as constriction material in structural engineering. These materials have proven themselves to be valuable for use in the construction of new buildings and bridges and for the upgrading of existing structures (Bank 2006).

3.1 Rehabilitation

Majority of rehabilitation works consist of repair of old deteriorating structures, damage due to seismic activities and other natural hazards. Structural strengthening is also required because of degradation problems which may arise from environmental exposure, inadequate design, poor quality construction and a need to meet current design requirement. Therefore, structural repair and strengthening has received much attention over the past two decades throughout the world (Karbhari *et al.* 2003). Therefore, there is an urgent need for development of effective, durable and cost-efficient repair, strengthening and retrofit materials and methodologies (Hollaway and Head 2001).

Generally, FRP composites can be utilized for structural rehabilitation in the following situations (Bakis *et al.* 2002):

- Deficiencies at the design stage, including: design errors, inadequate factors of safety, use of inferior class materials and poor construction quality.
- Change of use, in service, namely, increased safety requirements (upgrading of structural design standards), modernization that causes redistribution of stresses and increase of the applied load.
- Ageing of materials that compromise the load capacity of the structure: for example concrete degradation in hostile marine or industrial environments.
- Accidents, as fire or seismic events.

There are two possible alternatives to restore a deficient structure to the required standard; these are complete or partial demolition and rebuild, or beginning of a programme of strengthening (ACI 364.1R 1994).
Within the scope of rehabilitation of concrete structures, it is essential that differentiation is made between repair, strengthening and retrofit terms which are often erroneously used interchangeably (Hollaway and Head 2001):

- In “repairing” a structure, the FRP composite is used to fix a structural or functional deficiency such as a crack or a severally degraded structural component.
- The “strengthening” of structures is specific to those cases wherein the addition or the addition or application of the FRP composite would enhance the existing designed performance level.
- The term “retrofit” is used to relate to the seismic upgrades of facilities.

3.1.1 Repair and Strengthening

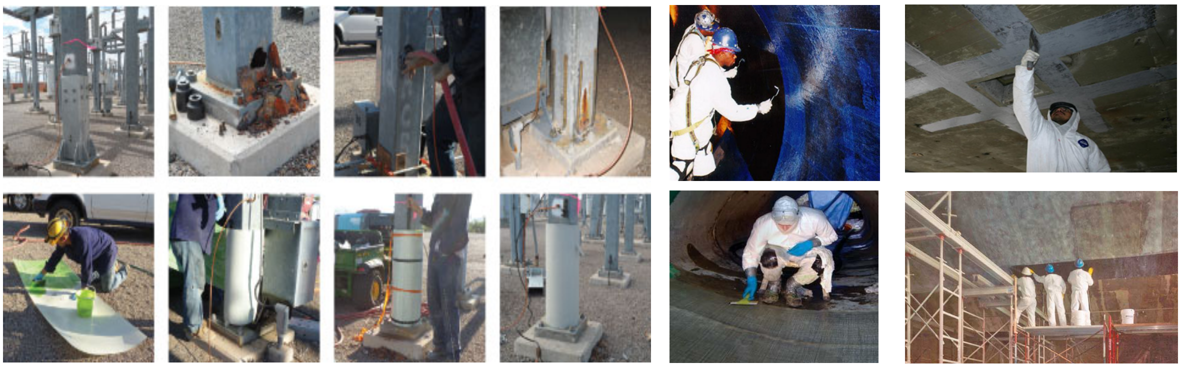

Figure 1: Repair and strengthening.

Repair with FRP composites has been used successfully on concrete, timber, metal and masonry structures. The predominant role of concrete as a structural construction material simulated the application of FRP composite in repairing of concrete structures, namely, bridges and large structural elements (Guideline No. 03742, 2006, Documents Scientifiques et Techniques, 2001).

The basic FRP strengthening technique, which is most widely applied, involves the manual application of either wet lay-up or prefabricated systems by means of cold cured adhesive bonding. Common in this techniques is that the external reinforcement is bonded onto the concrete surface with the fibers as parallel as practically possible to the direction of principal tensile stresses. Besides the basic techniques, several special techniques have been developed, namely the automated wet lay-up wrapping (of columns or chimneys, for example), use of pre-stresses FRP (to close open cracks in bridge decks, for example) (FIB Bulletin 14 2001). Near-surface mounted (NSM) technique may also be thought as a special method of reinforcement of concrete structures. In the NSM method, grooves are first cut into the concrete cover and the FRP reinforcement, usually a laminate strip, is bonded therein with appropriate groove filler, typically epoxy paste or cement grout.

3.1.2 Seismic Retrofit

The problem of structural deficiency of existing constructions is especially acute in seismic regions, as, even there, seismic design of structures is relatively recent. The enhancement of confinement in structurally deficient concrete columns in seismically active regions of the world has proven to be one of the most significant applications of FRP materials in infrastructure applications (FIB Bulletin 35 2006).

Seismic retrofit of reinforced concrete structures, namely bridges, using conventional steel techniques, whilst effective, has been found to be time consuming, cause significant traffic disruption, rely on field welding and is susceptible o corrosion. Additionally, many of the methods increase the stiffness and strength capacity of the columns putting adjacent structural elements at risk from higher transmitted seismic forces. The use of FRP composites in this application (figure 2), not only provides a means of confinement, without the associated increase in stiffness, but also enables the rapid fabrication of cost effective and durable jackets with little traffic interference.

Figure 2: Seismic retrofitting of a bridge with FRP composites.

3.2 Concrete Structures Reinforced with Fiber Composites

Concrete reinforced with fiber reinforced polymer (FRP) materials has been under investigation since the 1960's. The predominant role of concrete as a construction material and the problems associated with corrosion of steel reinforcement stimulated the development of fiber composites for internal (ACI 440.1R 2003, FIB Bulletin 40, 2007) and external (FIB Bulletin 14, 2001, ACI 440.2R 2002) reinforcement of concrete and pre-stressing cables and tendons (ACI 440.4R 2004). Unstressed FRP reinforcement has been developed in a number of forms including ribbed FRP rod similar in appearance to deformed steel reinforcing bar, undeformed E-glass and carbon fiber bar bound with polyester, vinylester or epoxy resin, E-glass mesh made from flat FRP bars and prefabricated reinforcing cages using flat bars and box sections (Gowripalan 1999). Stressed FRP reinforcement is also available, usually consisting of bundles of rods or strands of fiber-reinforced polymer running parallel to the axis of the tendon. These are used in a similar fashion to conventional steel tendons (Gowripolan 2000).

The durability performance of FRP reinforcements is considered by some (Gowripalan 1999, Ko *et al.* 1997) to offer a possible solution to the problem of corrosion of steel reinforcement, a primary factor in reduced durability of concrete structures. Other reported advantages of FRP rebar include enhanced erection and handling speeds (Cowie 1991) and suitability to applications which are sensitive to materials which impede radiowave propagation and disturb electromagnetic fields.

3.2.1 FRP bars, rods and grids

The use of FRP reinforcing bars and grids for concrete is a growing segment of the application of FRP composites in structural engineering for new construction (FIB Bulletin 40, 2007). For an effective reinforcing action, it is necessary to develop bond strength between FRP and concrete. This is attained in FRP rod by having various types of deformation systems, including exterior wound fibers, sand coatings and separately formed deformations.

FRP reinforcing bars and grids for concrete with both glass and carbon fibers are produced by a number of companies in USA, Asia and Europe (ACI 440R 2007). Their use has become recurrently and is no longer confined to demonstration project as in the past. Applications have become routine for certain specialized environments, namely in bridge decks and in underground tunnels.

3.2.2 FRP cables for prestressing and poststressing applications

Composite cable applications in the infrastructure are used in the construction of suspension and stay cables for bridges, pre-stressed tendons for various concrete structures and external reinforcements for structural beams. All these applications require materials that incorporate high tensile strength and, in addition, require characteristics such as corrosion resistance and light weight (Hollaway 2003).

Corrosion of steel pre-stressing tendons can lead to the concrete degradation and the deterioration of structural integrity. In cable-stay applications, both corrosion and fatigue make the replacement of conventional cables a significant life cost. FRP composites have good corrosion, durability and fatigue characteristics and therefore the utilization of these materials does make good engineering sense. The initial cost of the cables is higher than their competitors but this must be weighed against reduced transportation and handling costs, reduces maintenance and the anticipated longer useful life for individual stay cables and for concrete structures pre-stressed with FRP composite cables.

FRP cables are unidirectional reinforced structural elements made from glass, aramid or carbon fibers embedded in the polymer matrix. Different shapes exists, such as bars, cables, rectangular strips and braided reinforcement. Carbon fiber and aramid cables are used for pre-tension and post-tension concrete, however glass fiber cables are not recommended for pre-tension due to the low resistance to alkaline environments (Susana Cabral-Fonseca 2008).

3.3 New FRP Civil Structures

A small number of new load bearing civil engineering structures have been made predominantly from FRP materials over the last three decades. These include compound curved roofs (Hollaway 2002) pedestrian and vehicle bridges decks (FHWA 2002) energy absorbing roadside guardrails (Bank and Gentry 2000), building systems, modular rooftop cooling towers (Barbero and GangaRao 1991), access platforms for industrial, chemical and offshore (Hale 1997), electricity transmission towers, power poles, power pole cross-arms and light poles and marine structures such as seawalls and fenders (Weaver 1999).

FRP pultruded structures profiles have been used in a significant number of structures to data, including pedestrian bridges, vehicular bridges, building bridges, building frames, cooling towers, walkways and platforms, etc. (Susana Cabral-Fonseca 2008).

4. THE FUTURE

The future holds unlimited promise for the use of FRPs in structural engineering applications. One of the most exciting recent advances is the development of smart materials and smart structures. Smart are those in which sensors are installed to continuously monitor the performance of the structure throughout its lifetime. Recently, FRP materials have been developed which include fiber-optic sensors (FOS) as part of their internal structure. These FOS can be used to measure variations in strain and temperature within the structure itself, and can provide information to engineers on its short and long-term performance. These materials can be considered an emerging technology, although several smart structures have already been built in Canada and are currently under observation. Smart structures and materials will undoubtedly become more important and widespread in the future. Figure 3 gives an example of a smart structure in Canada: the Taylor Bridge near Winnipeg (Fitzwilliam 2006).

Figure 3: Taylor Bridge near Winnipeg in Canada

5. CONCLUSIONS

The application of FRPs in civil infrastructures is not uncommon anymore in many European and North American countries, and some Asian countries like Japan, China etc. FRP is now widely used in strengthening of existing structures and repair of building and bridges that were damaged, shows that this technique, in many cases, may be a superior alternative to traditional techniques both from technical and economical perspectives. So, it is very clear that the volume of repair and retrofit works with FRPs will increase substantially in the future. For new structures, the application of FRPs is very promising and will depend on the ability to compete with the conventional materials (e.g. steel and concrete). Lightness, structure and easy workability are three very positive aspects of FRP materials. However, design standards is a big factor, also FRPs exhibit anisotropic behaviour and their mechanical characteristics may be affected by the quantity and orientation of fiber reinforcement, temperature, environmental conditions and so on. The material costs of the FRP composites are several times more than that of the conventional materials. However, the life-cycle cost, including fabrication, application, protection and projected maintenance costs, is comparable and can be less than that of conventional materials. In the longer term, the challenge is to prepare a new generation of architects and civil engineers to utilize the full potential of FRP materials. This requires systematic education and training in the design and manufacture of FRPs for the construction industry.

REFERENCES

ACI 364.1R-94 (1994). Guide for Evaluation of Concrete Structures Prior to Rehabilitation. American Concrete Institute, Farmington Hills, MI.

ACI 440.1R-03 (2003). Guide for Design and Construction of Concrete Reinforced with FRP Bars. American Concrete Institute, Farmington Hills, MI.

ACI 440.2R-02 (2002). Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures. American Concrete Institute, Farmington Hills, MI.

ACI 440.4R-04 (2004). Prestressing Concrete Structures with FRP Tendons. American Concrete Institute, Farmington Hills, MI.

ACI 440R-07 (2007). Report on Fiber-reinforced Polymer (FRP) Reinforcement for Concrete structures. American Concrete Institute- Committee 440, Farmington Hills, MI.

ACI 440R-96 (1996). State-of-the-art Report on Fiber Reinforced Plastic Reinforcement for Concrete Structures. American Concrete Institute, Farmington Hills, MI.

ACI Committee-440 (2002). Guide for the Design and Construction of Externally Bonded FRP System for Strengthening Concrete Structures. American Concrete Institute, Farmington Hills, MI.

ACI Committee-503 (1992). Guide for the Selection of Polymer Adhesive with Concrete. ACI Mater J, 89(1):90-105.

Bakht, B., Al-Bazi, G., Banthia, N., Cheung, M., Erki, M.A., Faoro, M. (2000). “Canadian Bridge Design Code Provisions for Fiber-Reinforced Structures.” J Compos Const, ASCE, 4(1):3-15.

Bakis, C.E., Bank, L.C., Brown, V.L., Cosenza, E., Davalos, J.F., Lesko, J.J., Machida, A., Rizkalla, S.H., Triantafillou, T.C. (2002). “Fiber-Reinforced Polymer Composites for Construction – State-of-the-art Review.” Journal of composites for Construction, 6:2, 73-87.

Bank, L.C. (2006). Composites for Construction – Structural Design with FRP Materials. John Wiley & Sons, Inc.

Bank, L.C. and Gentry, T.R. (2000). “Composite Materials for Roadside Safety Structures.” Proceedings of ACUN-II International Composites Conference, University of New South Wales, Australia.

Bank, L.C., Gentry, T.R., Thompson, B.P., Russell, J.S. (2003). “A Model Specification of FRP Composites for Civil Engineering Structures.” Constr Build Mater, 17:405-37.

Barbero, E., GangaRao, H.V.S. (1991). “Structural Applications of Composites in Infrastructure.” SAMPE Journal, Vol. 27(6), SAMPE, USA.

Bousselham, A. and Chaallal, O. (2004). “Shear Strengthening Reinforced Concrete Beams with Fiber-Reinforced Polymer: Assessment of Influencing Parameters and Required Research.” ACI Struct J, 101(2):219-27.

Cowie, J.M.G. (1991). Polymers: Chemistry & Physics of Modern Materials, Blackie Academic Professional.

Cripps, A. (2002). Fiber-Reinforced Polymer Composites in Construction. CIRA C564.

Deniaud, C. and Cheng J.J.R. (2001). “Review of Shear Design Methods for Reinforced Concrete Beams Strengthened with Fiber Reinforced Polymer Sheets.” Can J Civil Eng, 28:271-81.

Documents Scientifiques et Techniques (2001). Réparation et Renforcement des Structures en Béton au Moyen des Matériaus Composites – Recommandations Provisoires. Association Française du Génie Civil.

Einde, L.V.D., Zhao, L., Seible, F. (2003). “Use of FRP Composites in Civil Structural Application.” Constr Build Mater, 177:389-403.

Engindeniz, M., Kahn, L.F., Zureick, A.H. (2005). “Repair and Strengthening of Reinforced Concrete Beam-Column Joints – State-of-the-art.” ACI Struct J, 102(2):187-97.

FHWA (2002), “Fiber Reinforced Polymer Composites Bridge Technology”, US Department of Transport Federal Highway Administration. http://www.fhwa.dot.gov/bridge/frp/index.htm

FIB Bulletin 14 (2001). Externally Bonded FRP Reinforcement for RC Structures. Fédération Internationale du Béton.

FIB Bulletin 35 (2006). Retrofitting of Concrete Structures by Externally Bonded FRPs with Emphasis on Seismic Applications. Fédération Internationale du Béton.

FIB Bulletin 40 (2007). FRP Reinforcement in RC Structures. Federation Internationale du Beton.

Gowripalan N. (1999). “Fiber Reinforced Polymer (FRP) Applications for Pre-stressed Concrete Bridges.” Proceedings of ACUN 1 Conference Composites: Innovations and Structural Applications, UNSW, Australia.

Gowripolan, N. (2000). “Design Considerations for Pre-stressed Concrete Beams with Fiber Reinforced Polymer (FRP) Tendons.” Fiber Composites: Who Needs Them? A Workshop for Civil and Structural Engineers, University of Southern Queensland, Australia.

Guideline No. 03742 (2006). Guide for the Selection of Strengthening Systems for Concrete Structures., International Concrete Repair Institute.

Hale, J. M. (1997). “Strength Reduction of GRP Composites Exposed to High Temperature Marine Environments.” Proceedings of ICCM11 International Conference on Composite Materials. Australian Composites Structures Society, Australia.

Halliwell, S.M. (2000). Polymer Composites in Construction. BRE Center for Building Fabric, CRC Ltd.

Hollaway, L. (2002). “The Development and the Future of Advanced Polymer Composites in the Civil Infrastructure.” Proceedings of the International Conference on Advanced Composites in Construction. University of Southampton, http://www.cosacnet.soton.ac.uk/ACIC2002/Monday/m0930Hollaway.pdf

Hollaway, L.C. (2003). “The Evolution of and the Way Forward for Advanced Polymer Composites in the Civil Infrastructure.” Construction and Building Materials, 17,365-378.

Hollaway, L.C. (2003). “The Evolution of and the way Forward for Advance Polymer Composites in the Civil Infrastructure.” Constr Build Mater, 17:365-78.

Hollaway, L.C. and Head, P.R. (2001). Advanced Polymer Composites and Polymers in Civil Infrastructure. Elsevier.

Karbhari V.M. (2001). “Materials Consideration in FRP Rehabilitation of Concrete Structures.” J Mat Civil Eng, ASCE,13(2):90-7.

Karbhari, V.M. Ed. (2007). Durability of Composites for Civil Structural Applications. Woodhead Publishing Materials, USA.

Karbhari, V.M., Chin, J.W., Hunston, D., Benmokrane, B., Juska, T., Morgan, R., Lesko, J.J., Sorathia, U., Reynaud, D. (2003). "Durability Gap Analysis for Fiber-Reinforced Polymer Composites in Civil Engineering." Journal of Composites for Construction, ASCE, 7:3,238-247.

Ko, K., et al. (1997). "Fiber Architecture based Design of Ductile Composite Rebar for Concrete Structures." Proceedings of ICCM-11: International Conference on Composite Materials. Australian Composites Structures Society, Australia.

L.A. Bisby and J. Fitzwilliam (2006). ISIS Educational Module 2: An Introduction to FRP Composites for Construction, Prepared by A Canadian Network of Centers of Excellence, www.isiscanada.com.

Lopez-Anido, R., Karbhari V.M.Ed. (2000). Emerging Materials for Civil Infrastructure – State-of-the-Art. American Society of Civil Engineering, USA.

Lu, X.Z., Teng, J.G., Ye, L.P., Jiang, J.J. (2005). "Bond-Slip Models for FRP sheet/Plates Bonded to Concrete." Eng Struct, 27:920-37.

Meier, U. (1992). "Carbon Fiber Reinforced Polymers: Modern Materials in Bridge Engineering." Struc Eng Int, 1:7-12.

Mufti, A.A. (2003). "FRPs and FOSs Lead to Innovation in Canadian Civil Engineering Structures." Constr Build Mater, 17:379-87.

Mufti, A.A., Erki, M.A. and Jaeger, L.G. (1991). "Advanced Composite Materials with Application to Bridges." Canadian Society for Civil Engineering, pp.297.

Mufti, A.A., Labossiere, P., Neale, K.W. (2002). "Recent Bridge Application of FRPCs in Canada." Struct Eng Int, 2:96-8.

Nanni, A. (1993). "Flexural Behavior and Design of RC Members Using FRP Reinforcement." J Struct Eng, ASCE, 119(11):3344-58.

Nanni, A. (2003). "North American Design Guidelines for Concrete Reinforcement and Strengthening Using FRP: Principles, Applications and Unresolved Issues." Constr Build Mater, 17:439-46.

Saadatmanesh, H. (1994). "Fiber Composites for New and Existing Structures." ACI Struc J, 91(3):346-54.

Susana Cabral-Fonseca (2008). "Fiber Reinforced Polymer Composite Materials Used in Civil Engineering", TR 6.2, Technical Report, Smart & Green Structural and Repair Materials.

Uomoto, T., Mutsuyoshi, H., Katsuki, F., Misra, S. (2002) "Use of Fiber Reinforced Polymer Composites as Reinforcing Material for Concrete." J Mat Civil Eng, ASCE, 14(3):191-209.

Weaver, A. (1999). "Composites: Worldwide Markets and Opportunities", Materials Today, Available March 2003: http://www.materialstoday.com/

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TEST OF MASONRY WALLS FOR DIFFERENT STRENGTHENING TECHNIQUES

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ABSTRACT

This paper investigates strengthening masonry shear walls using two types fiber reinforced polymer (FRP) sheets. An experimental research program is undertaken. Masonry wall made from wire-cut clay brick specimens are tested strengthening by PET (Polyethylene Terephthalate) and CFRP (Carbon fiber reinforced polymer) sheets. Strengthening is considered on both side of the wall to ensure uniformity and symmetrical stiffness of the wall. Static tests are carried out on seven masonry panels, under a combination of vertical preloading, and in-plane horizontal shear loading. The mechanisms by which load was carried were observed, varying from the initial, uncracked state, to the final, fully cracked state. The results demonstrate that a significant increase of the in-plane shear capacity of masonry can be achieved by bonding with PET and CFRP sheets to the surface of masonry walls. The experimental data were used to assess the effectiveness of the strengthening of one FRP over the other and also the percent of gross area covered by the FRP sheets. Subsequently suggestions are made to allow the test results to be a reasonable source of guideline in the design of strengthening for masonry structures.

Keywords: Brick; Masonry; Shear wall; FRP sheets; Static Shear tests.

1. INTRODUCTION

Externally bonded Polyethylene Terephthalate (PET) fiber with a large fracture strain is one of the retrofitting technique that has drawn a significant attention as an unique alternative to CFRP or GFRP due to its pronounced ductile behavior and relatively low material cost (Figure 1), but not compromising the other advantages of FRP like low weight-strength ratio, short installation period and a minimum intervention during the fabrication process. The main objective of using FRPs is to enhance performance of structure at normal loading condition and to offer greater resistance at the time of severe loading. Neither too much stiffness nor the very high strength will be coherent with the overall performance of the masonry structures. Material with high stiffness will produce very little deformation before complete failure and the failure if any will be brittle and explosive in nature without any prior warning, which is not the expected failure mode from a well performed structure. On the other hand too soft material with nominal strength will not be well fitted with the purpose of strengthening. A variety of approaches has previously been undertaken to investigate masonry strengthened with FRPs. Masonry wall strengthening by GFRP (Stratford et al. 2004) shows quite ductile behavior prior to failure and a significant increase of shear load capacity (65%) where as the failure was primarily for the debonding of the GFRP sheet. Another technique of strengthening is done by mounting the FRP rod inside the horizontal mortar joint and is embedded near the surface as shown in Figure 2a. (Tinazzi et al.2000; Tumialan et al. 2001). Alternative to this system is to attach the

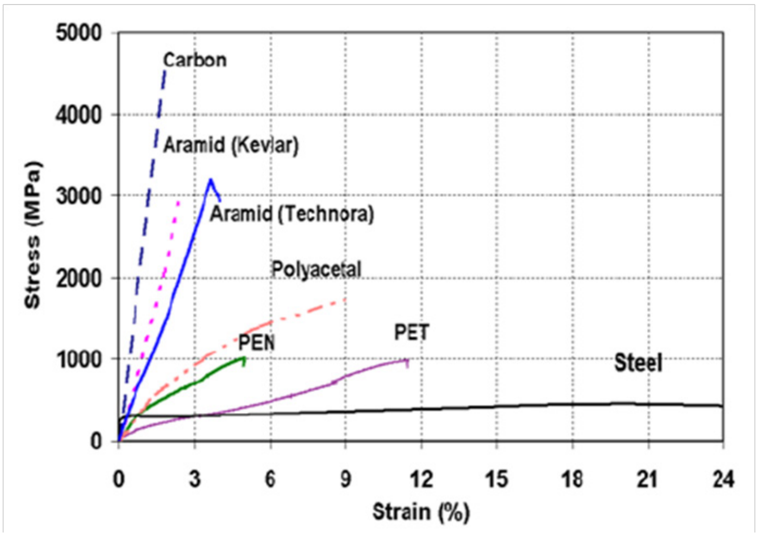
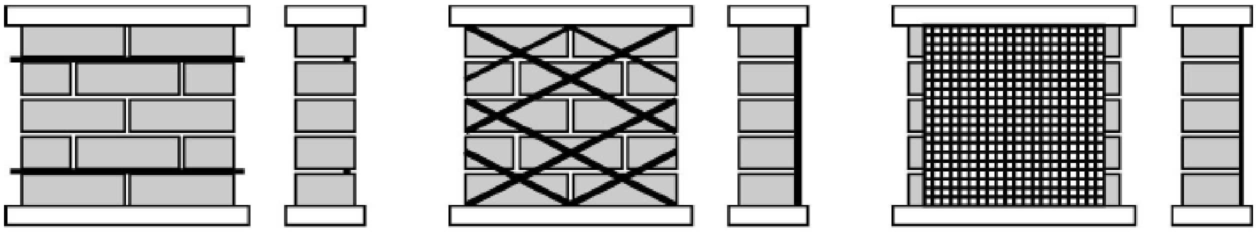


Figure 1: Tensile strength of different FRPs



(a) Near surface mounted FRP bars (b) Strip FRP in diagonal fashion (c) Sheet FRP over the wall

Figure 2: Different techniques of FRP installation on masonry wall walls strength of different FRPs

FRP strips over the surface of the wall in a diagonal fashion (Schwegler 1994). The FRP strip act as a truss made from unidirectional fibers (Figure 2b).
In this study, PET and CFRP have been used as strengthening materials. CFRP has a high stiffness among the other widely used FRPs, where as PET posses a relatively low stiffness but with a higher fracturing strain than CFRP. The purpose of this study is to show the difference in behavior of masonry shear wall for two distinct FRPs. Ultimate load bearing capacity, deformation at peak load, mode of failures, are observed in this study for different arrangement of the FRPs.

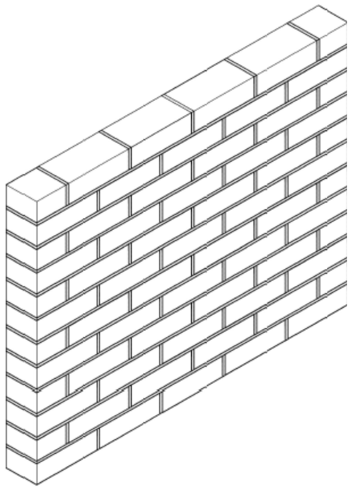
2. METHODOLOGY

For Different loading schemes for testing of masonry shear wall can be found elsewhere in the literatures. TNO 2004 has made an inventory of different test methods with different loading and boundary conditions. In this experimental study, a total of 7 walls having a nominal dimension of 1200 × 1000 × 120 m were constructed to test for shear. All of the walls were fabricated with single layer stretcher bond of bricks having a dimension of 240×120×66 mm with a compressive strength of 17MPa. A 10 mm thick mortar with a compressive strength of 12 MPa were used throughout. Table 1 gives the detail of the walls. In this table RW stands for reference wall, PW for wall strengthening with PET sheet and CW for wall strengthening with CFRP sheet.

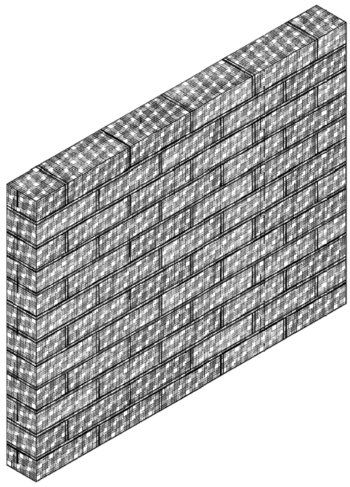
Table 1: Detail of masonry specimen with different FRP orientations

Wall ID	Properties							
	Vertical compression f_v (MPa)	Brick strength, f_c^b (MPa)	Mortar strength, f_c^m (MPa)	Area Covered by FRPs: PET(%)	Area Covered by FRPs: CFRP (%)	L/H	FRP orientation	Peak load kN
RW2	0.25	17	12	_	_	1.25	--	30.0
PW1	0.25	17	12	100	_	1.25	all over	92.0
PW2	0.25	17	12	40	_	1.25	checkered	90.0
PW3	0.25	17	12	20	_	1.25	diagonal	62.0
CW1	0.25	17	12	_	100	1.25	all over	107.0
CW2	0.25	17	12	_	40	1.25	checkered	99.0
CW3	0.25	17	12	_	20	1.25	diagonal	94.0

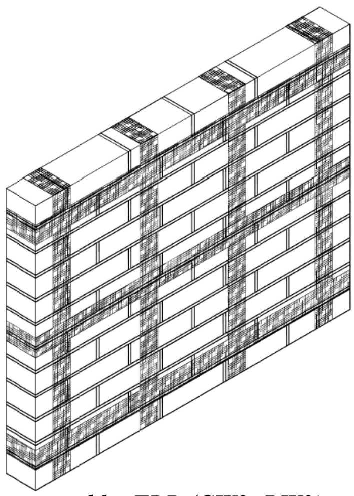
2.1 Specimen preparation



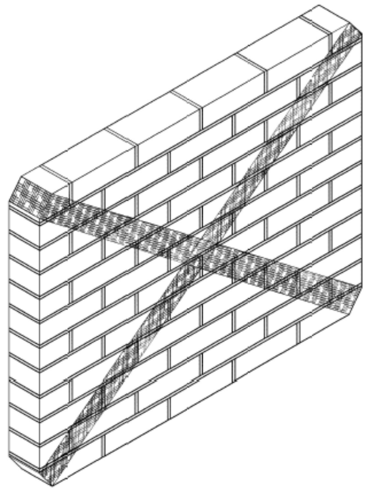
(a) Control wall(RW2)



(b) 100% area covered by FRP (CW1, PW1)



(c) 40% area covered by FRP (CW2, PW2)



(d) 20% area covered by FRP (CW3, PW3)

Figure 3: Wrapping technique of masonry wall by FRP waspecimen

PET and CFRP sheet both having unidirectional fiber was applied on wall specimens in three different fashions as shown in Figure 3. The FRPs were applied in wet layup procedure as follows:

Table 2: properties of FRPs and resign

FRP properties	*PET 600*	*CFRP* FTS-C1-20	Properties	*RESIN (D-90 R)*	*PUTTY (T-30)*
Fracture Strength (N/mm^2)	740	3400	Compressive strength, MPa	85	-
Elastic Modulus (kN/mm^2)	10±1	245	Flexural strength, MPa	65	-
Elongation (%)	10±1	1.5	Tensile strength, MPa	45	20.0
Thickness (mm)	0.841	0.111	Compressive modulus, MPa	2500	-
*Width (mm)	300	250	Tensile modulus, MPa	1560	-

	Elongation after fracture, %	28%	-

the wall specimens were cleaned from loose mortar and dirt with the help of a wire brush. Epoxy putty (filler) was used to fill the depression onto the wall that offered a smooth and plane surface ready for FRP laying. A thin layer of primer was then applied all over the wall to seal pores in the masonry so that the subsequent application of epoxy resin was not absorbed. Epoxy resin having the properties mentioned in Table 2 was then applied over the wall where it was necessary. FRP sheets were cut in accordance with the length needed for the walls to cover the 100%, 40% and 20% of the wall gross area (see Fig. 3). Resin was then applied with the help of a roller brush all over the FRP strip so that it was fully saturated by the resin. As soon as the application of the resin was completed, the FRP strips were laid over the wall and wrapped tightly to keep them in place. For diagonal strip, the wall corners were cut in same dimension of the width of the strip to avoid sharp edges and problem in wrapping.

2.2 Test setup

After the required curing period, the wall was transferred to the testing frame. The wall top and bottom were anchored with the top and bottom channel beams with the help of six 22 mm bolts as shown in Figure 4. In between wall and the side of the channel beam, a steel plate of 16mm thick was inserted so that the plate creates tremendous pressure on the wall through the tightening of the bolts. Paris plaster was also used to offer a flat surface between wall and steel plate.

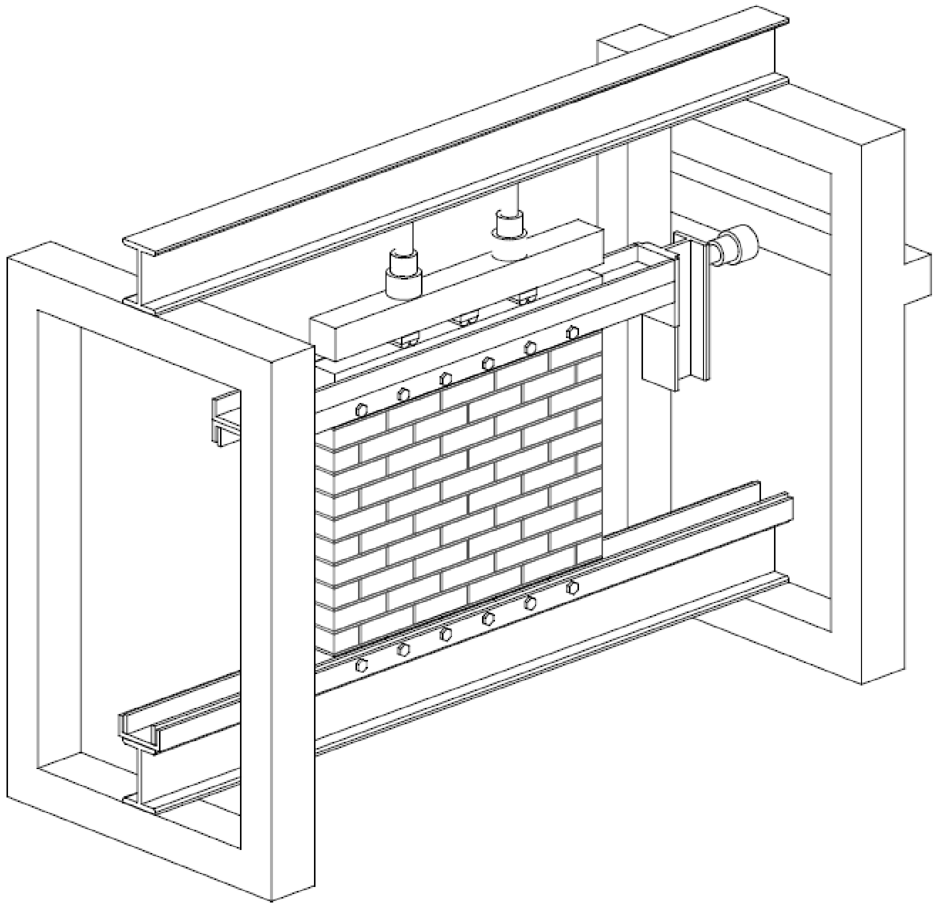


Figure 4: Experimental setup for a typical shear wall specimen

A pre-compression of 40 kN which was equivalent to uniform pressure of 0.25MPa was applied on the top of the wall through two hydraulic jacks, ahead of the shear loading to simulate the load of structural component coming on the wall from the above in the real world. Sufficient wire strain gages (KYOWA-5) and LVDTs were used to record all the necessary information during the course of testing. Loads were applied on roller and ball-bearing system to avoid any excessive friction and deviation of loading points. The lateral load was then applied with the help of a hydraulic jack having a capacity of 50 tons. Remote pump was used to increase the lateral load gradually until complete failure of the masonry wall was ensured. All of the data (displacement, strain, load) were accumulated through a digital data logger.

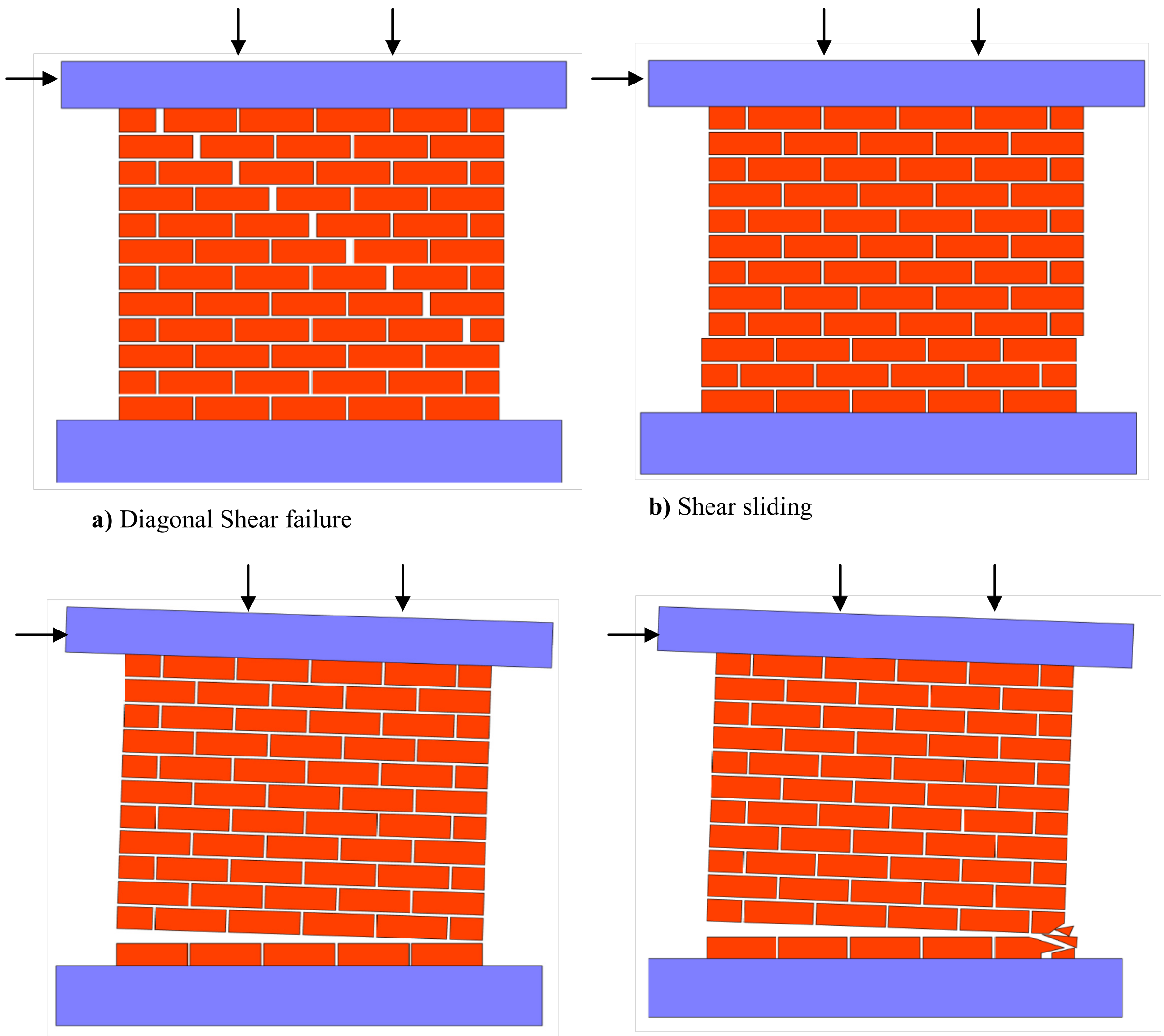
3. TEST RESULTS AND DISCUSSION

The entire shear wall specimens were tested consecutively one after another. Static lateral load was applied gradually and increased at a constant rate of 10kN/min until failure. Application of load was paused intermittently to observe the crack and damage if any, on the wall. The crack patterns, failure mode, lateral load-deflection response has been discussed in the following sections.

3.1 Mode of Failure of Unreinforced Masonry Wall (URM)

The lateral shear strength of the reinforced masonry walls depends on the failure mode of controlling wall, which is governed by a number of variable parameters, they are,
Masonry aspect ratio, L/H
Vertical compression on masonry, P
Masonry compressive strength, f_m' (which is a function of unit and mortar strengths)
Masonry tensile strength, f_{tm}
Masonry shear strength, τ_u
Masonry elastic modulus, E_m
Masonry shear modulus, G_m

These variables control the inelastic mode of failure of masonry shear walls. The lateral strength of unreinforced in-plane masonry walls is limited by diagonal tension, bed-joint sliding, toe crushing, or rocking (Fig. 5). Among them rocking and bed-joint sliding are classified as deformation controlled actions because lateral deflections of walls and piers can become quite large as strengths remain close to constant. Diagonal tension and toe crushing are classified as force-controlled actions because they occur when a certain stress is reached, and can cause sudden and substantial strength deterioration Stair-stepped diagonal cracking can also be considered as a deformation controlled action because frictional forces along bed joints are conserved with vertical compressive forces. However, diagonal tension must be classified as a force-controlled action unless stair-stepped cracking can be distinguished from diagonal cracking through units. (FEMA 356, 2000). It should be noted that not all the above failure modes will involve collapse of the wall panels and the final failure may be the combination of several failure modes. However, a rationally developed failure criterion should be able to predict the tensile, compressive and shear type of failure (Zhuge et al. 1998). A more general way to represent the various failure mode of masonry can be seen in Fig. 6, where the interaction between normal and shear stress on masonry bed joint are shown.



a) Diagonal Shear failure

b) Shear sliding

c) Flexural cracking

d) Flexural cracking with toe crushing

Figure 5. Different Failure mode of URM

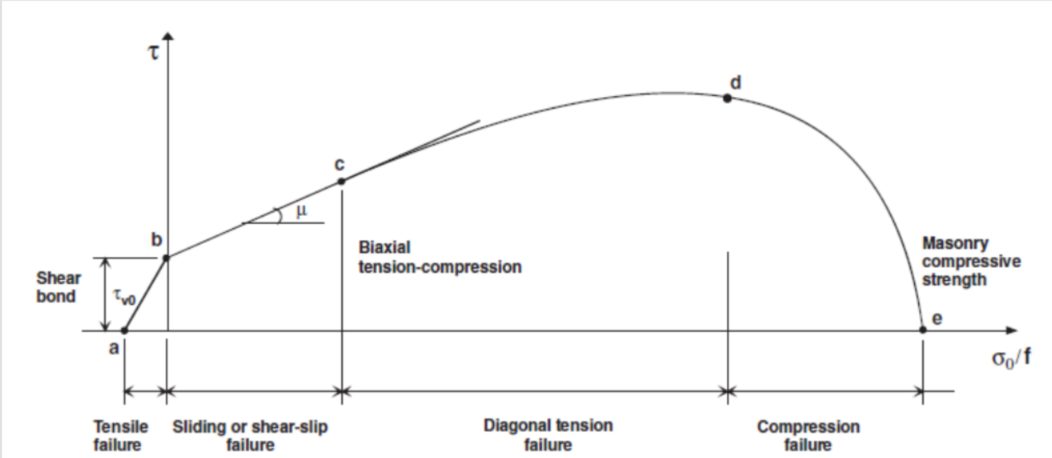


Figure 6. Behavior of URM under combind shear and normal stress [Drysdale et.al. (1994)]

Masonry Non-liner Behavior

Nonlinear behavior of URM walls has been shown to be dependent on the aspect ratio (*L/h*) and the amount of vertical compressive stress. In the following section, an exhaustive discussion on various failure modes and their corresponding mechanisms has been given.

(*L/H* > 1.5): Behavior of relatively stocky walls is typically governed by diagonal tension or bed joint sliding, depending on the level of vertical compression, masonry tensile strength, and bed-joint sliding shear strength.

(1.0 < *L/H* ≤ 1.5): In walls with a moderate aspect ratio, considerable strength increases have been observed after flexural cracks form at the heel of a wall as the resultant vertical compressive force migrates towards the compressive toe. Results from experiments by Epperson and Abrams (1992) and Abrams and Shah (1992) have revealed these tendencies.

(*L/H* < 1.0): For more slender walls loaded with a relatively light amount of vertical compressive force, flexural cracks will develop along a bed joint near the base of the wall. When the lateral force approaches a value of *PL/2h*, the wall will start to rock about its toe, provided that the shear strength will not be reached. Again, flexural tension strength at the wall heel does not limit lateral strength. (FEMA 274, 1997)

Mode of failure of FRP Reinforced Wall

When FRP was bonded to the surface of the wall, compressive crushing type of failure are quite common (Hamid et al. 2005). Also FRP premature debonding or fracture was commonly observed during the testing and in general FRP could not reach its ultimate strength (Ehsani et al. 1997; Stratford et al. 2004; ElGawady et al. 2007). Experimental tests indicated that the failure pattern was affected by the strength, orientation, amount and anchorage length of FRP. (Ehsani et al. 1997; Hamid et al. 2005).

3.2 Observation of Crack Pattern

3.2.1 Control wall (RW2)

Initially for quite some time, no crack was observed anywhere on the wall. When the load reach about 23kN, a flexural crack just appeared on the wall heel (loading side) at about one third height of the wall, consequently the load dropped to 19 kN. As it was mentioned earlier that wall of this aspect ratio (*L/H > 1*) is governed by a force-controlled action, the load will further increase even after the appearance of initial flexural crack. With the increase of load, the crack further propagated almost halfway of the length of the wall and there after took a turn to downward in a stepped fashion and travelled all the way to the wall toe. A faster crushing on the wall toe caused a rapid decreased in load (Fig.7).

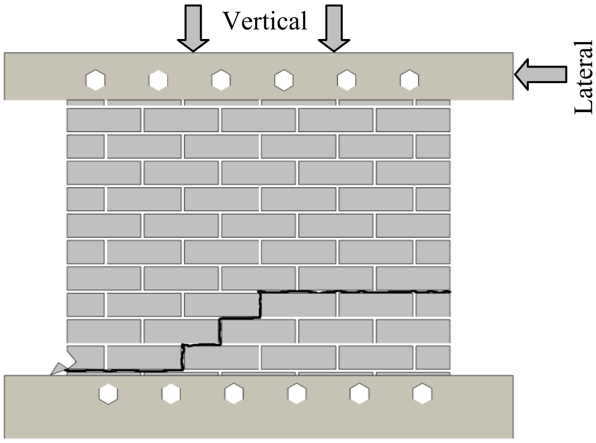
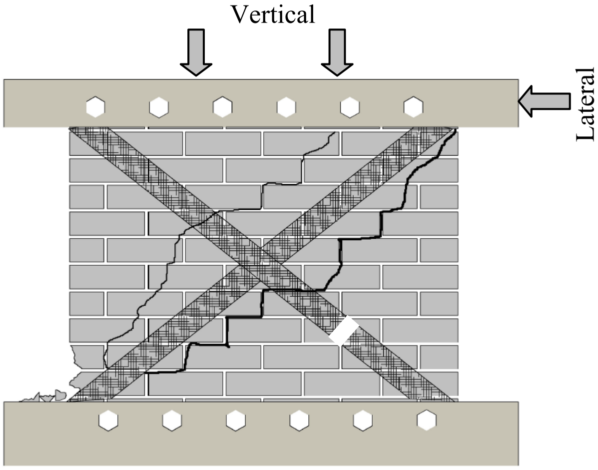


Figure 7: Observed crack pattern in RW2

3.2.2 FRP Reinforced wall

Wall with Diagonal CFRP Strap (CW3): It is interesting to note that initially the stiffness of the wall was quite similar to that of reference and remained same until the load reached about 75% of the peak load. At this point some fine line cracks appeared at the top of the wall and just below the compression CFRP strap on loading side and start to propagate downward through the bricks in a stepped fashion. The crack could not travel further and was arrested at the mid height of the wall by the diagonal tension strip of CFRP. Another crack almost parallel and above to this one stated to propagate downward in the same fashion with a lesser crack opening displacement. In between these two cracks, an effective compression strut can be seen. Once the load increased to its maximum, a sudden rupture of the diagonal tension strip of the both sides of the wall happened and the load suddenly dropped to 50% of the peak load. At that time the crack became wider and the crack opening increased remarkably (Fig. 8).

Wall with Diagonal PET Strap (PW3): The initial stiffness was quite similar to that of CW3 and remains almost same until it attained a load of 50kN. During that time some crack appeared at the mid height of the wall on loading side and propagate towards the bottom in a stepped fashion cracking both brick units and mortar joints . Another crack appeared almost at the same time along the center line of the wall on the wall top and traveled downward and branching towards the bottom and towards the wall toe. Debonding of the diagonal tension strip was seen at the upper half of the wall and that make it easy for the crack to travel further downward (Fig. 9).



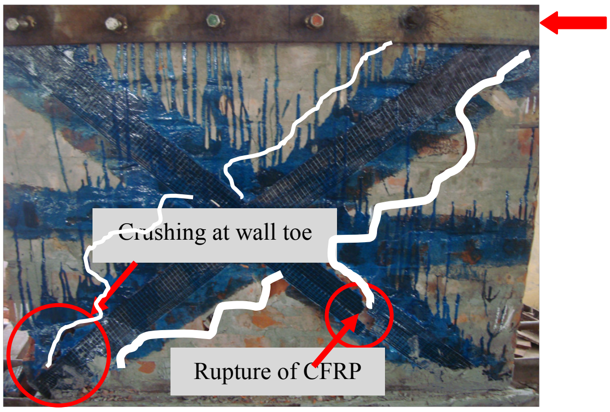


Figure 8. Observed crack pattern on CW3

Wall with Grid CFRP Strap (CW2): This wall was strengthened by CFRP strip of 70 mm width in a cheakard fashion as shown in Fig. 3(c) which covers approximately 40% of the wall gross area. The initial stiffness was much greater than the unreinforced wall RW2. Attaching CFRP on 40% area of the wall made it too stiff for lateral deformation and the whole wall behaves as a stiffened composite system. The stiffness did not changed until the load goes about 70% of the peak load. Only after that a flexural crack along the wall heel appeared at a load of 80 kN. With the increase of displacement through the jack, the load increased a little but the crack

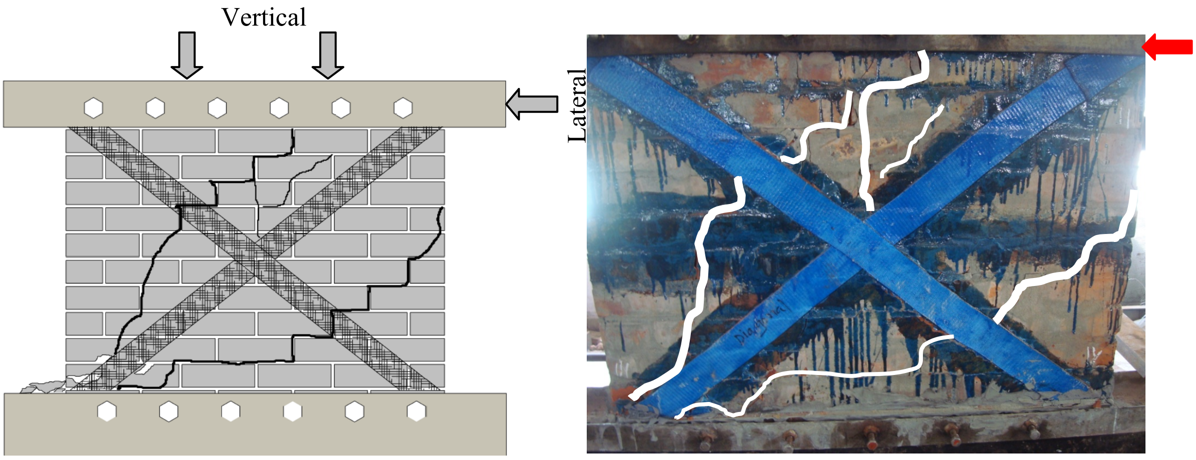


Figure 9. Position and pattern of cracks on PW3

opening widened further. After a little while another flexural crack appeared at the mid height of the wall on the loading side. A hair line crack appeared simultaneously at the wall center along the compression diagonal, as shown in Fig. 11. The load did not increase further after that and with increase displacement toe crushing happens and the load gradually felled to a minimum without the rupture of any CFRP strip. Some debonding phenomenon can also be noticed somewhere else on the wall.

Wall with Grid PET Strap (PW2): The wall PW2 strengthened with PET shows less stiffness than its counterpart of CFRP. The initial stiffness traced the same way of that of CW2 but only for a while then it shows little flexible than CW2. At a load of 63 kN the stiffness reduced to some extent due to appearance of flexural crack at the mid height of the wall on loading side as can be seen in Fig. 10.

Fully CFRP Sheet Wrapped wall (CW1): This shear wall was strengthened with 100% wrapping with CFRP. For this wall the stiffness as well as lateral load capacity was so high that the grip at the wall bottom prematurely failed before the wall attained its full strength. Fig. 11 shows that part of wall bottom is coming out of the anchorage.

Fully PET sheet Wrapped wall (PW1): The initial stiffness of this wall is very high due to higher percentage of PET has been used and remained unchanged until it reached about 60% of its peak shear capacity. Only after that the stiffness shows a softening tendency but no damage or crack whatsoever was seen on the wall, as the wall was fully covered by the PET sheet. At some point it was observed that the load is not increasing with the applied lateral displacement. Once it reached to its full strength, with further displacement, the load only descended. All of a sudden the wall moved from the horizontal loading jack with a huge jerk. The experimental was ceased at that point and was impossible to continue as both the wall and jack were out of plane as shown in Fig.12. Since the wall was fully wrapped by the PET sheet, damage if any could hardly be detected by visual observation.

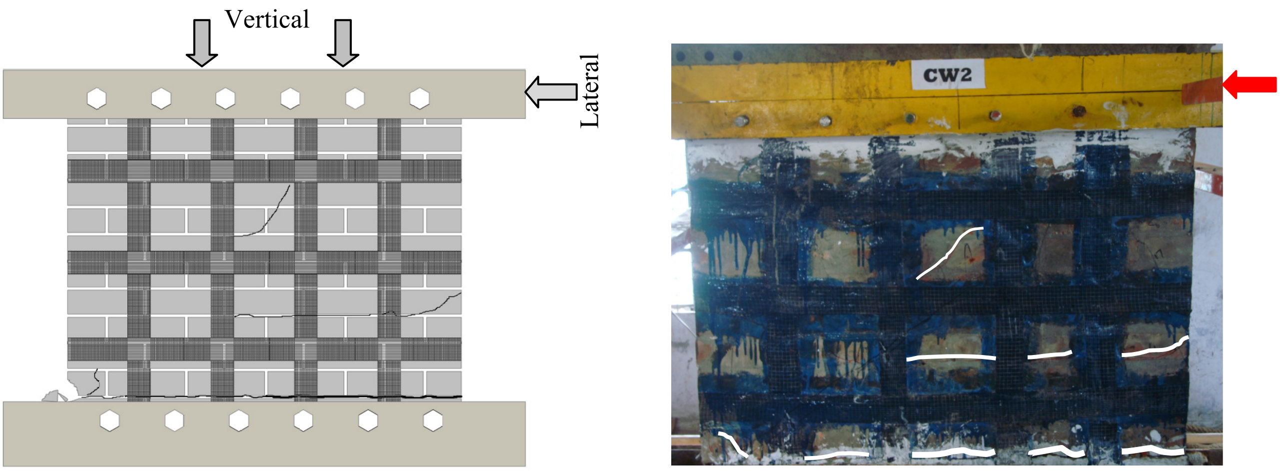


Figure 10: Position of Cracks on CW2

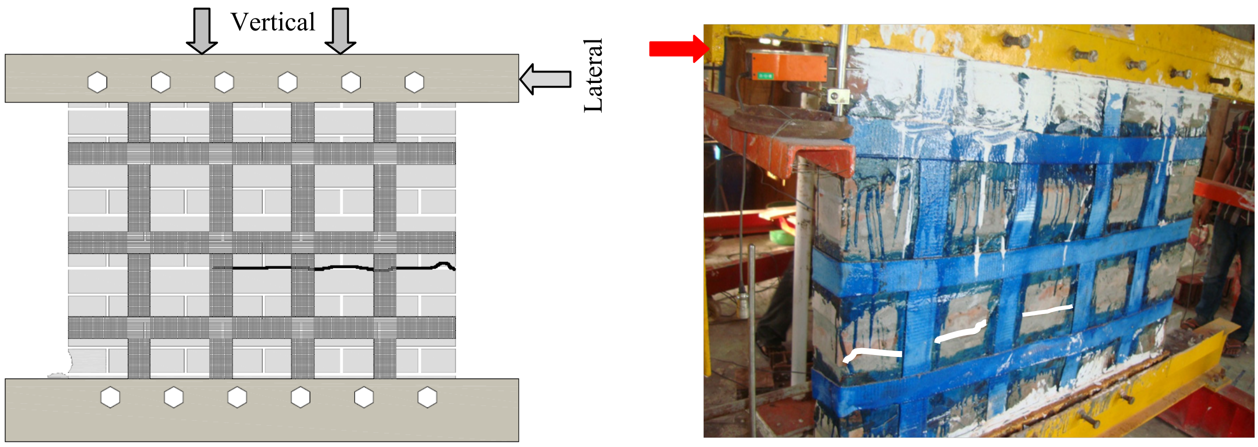


Figure.11. Position of Cracks in PW2

Figure 12. Premature failure of CW1 due to slip of anchorage at wall bottom.

Figure 13. Out-of-plane movement of wall PW1

3.3 Load Deflection Response

The load-deflection response of the entire masonry panels are plotted in Fig. 15. For CW# and PW3, the initial stiffness is quite similar to each other and remained unchanged until they reached about 70% and 80% of their full capacity respectively. In both of these two walls though the amount of FRPs used was the same, they had the diiferent shear capacity. CW3 had a capacity 3 times more than the unstrengthened wall (RW2) where as PW3 had two time more than URM wall.. For the case of PET, the strip didn't undergo delimitation or damage and produced large deformation without compromising the strength very much and for the case of CFRP wall, the strips gone sudden rupture and the strength fell to half of the full capacity. These two phenomena give a notion that the unlike CW3, PW3 wall which is reinforced with PET behaves in a more ductile way without catastrophic failure and can sustain a good amount of shear for quite a large deformation. This property of masonry is very much needed for masonry building in seismically active zone. Masonry wall that normally have a brittle nature of failure is quite catastrophic and does not show any sign prior to failure at the time of earthquake. Wall reinforced with CFRP strip on the other hand is much stiffer than its counterpart with PET strip and fail all of a sudden without much warning and time. Strengthening with CFRP is good for normal loading condition but quite detrimental at the time of earthquake tremor. It was also noticed that the CFRP strip have no but a little bending stiffness where as the PET strip is quite strong against bending. So, this also can be concluded that during the earthquake motion if a masonry wall move out of plane, CFRP cannot hold it in place where as PET can hold the wall in position for quite a long time.

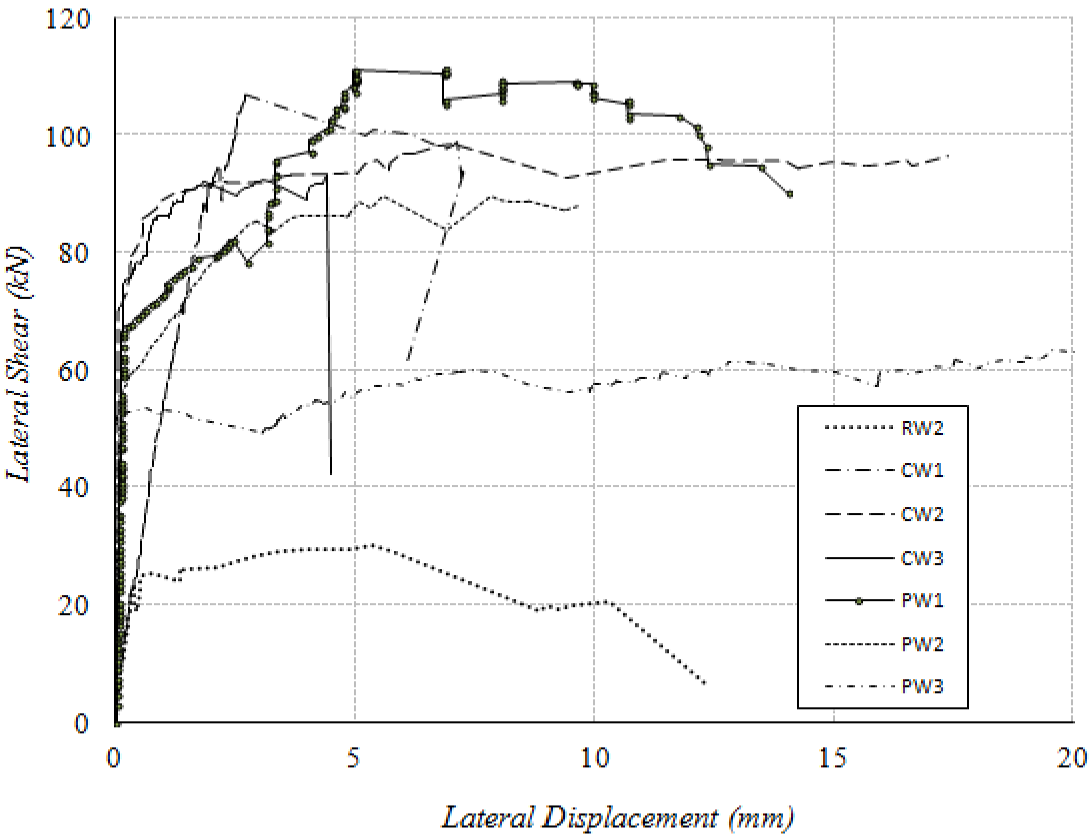


Fig. 14. Load displacement response of all of the masonry shear wall specimens

4. CONCLUSIONS & RECOMMENDATIONS

This experiment was carried out considering the following boundary conditions:

The two vertical loads equal in magnitude were applied at the wall top prior of lateral load, but were difficult to kept constant during the loading process due to the rotation of the upper beam on the wall. This rotational movement caused a pressure in the vertical jacks and thus increased the vertical pressure to some extent.

Since the shear capacity of the masonry panel increases with the increase of vertical load following the Mohr-Coulomb failure criteria, the increased vertical pressure in this experiment definitely increased the lateral load to some unknown quantity but could not be verified explicitly. ASTM E 72-61 suggests the standard racking test, where the vertical load N cannot be controlled if the rotation of the beam is restrained. So, this experiment is aligned with some standard where the vertical load is not controlled. On the other hand if the vertical load is kept constant throughout the loading process, the rotation of the top beam must be allowed to rotate to obtain the equilibrium.

The FRP was installed on the wall panel before the vertical load, whereas in reality the weight of the building above the wall exists as vertical pressure prior of the application of FRP.

Reinforcement of the masonry shear wall by both the PET and CFRP increases the shear capacity to a great extent for different FRP orientation. If the FRP is applied in diagonal fashion that covers only 20% of the panel area, the shear capacity increases twice for PET sheet and thrice for CFRP sheet as given in Table 1 and Figure 5. Whereas for the case of checkered fashion where the FRP covers 40% of panel area, the shear capacity increase about three times for both of the FRPs. When the penal is fully covered by FRPs, the capacity increases to a tremendous amount (five times for PET and 3.5 times for CFRP). Considering all of the cases, it can be concluded that PET offers more effective strengthening than CFRP sheet where strength is not the only considering criteria. The ductility of the panel is of great concern during earthquake and other natural disaster. PET offer greater ductility prior to failure than CFRP. When the cost of the material is also considered, PET is cheaper than the CFRP.

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REFERENCES

Stratford T, Pascale G, Manfroni O and Bonfiglioli B (2004). Shear strengthening masonry panels with sheet glass-fiber reinforced polymer. ASCE Journal of Composite for Construction. Vol.8, No.5, October 1, 2004, pp. 434-443.

Schwegler, G. (1994). Verstäken von mauerwerk mit faserverbundwerkstoffen in seismisch gefährdeten zonen. Bericht Nr. 229, PhD thesis, EMPA, Dübendorf, Germany.

Tinazzi, D., Modena, C., and Nanni, A. (2000). Strengthening of masonry assemblages with FRP rods and laminates. Proc. Int. Meeting on Composite Materials, PLAST 2000, Advancing with Composites, I. Crivelli-Visconti, ed., Milan, Italy, 411–418.

Tumialan, G., Huang, P.-C., Nanni, A., and Silva, P. (2001). Strengthening of masonry walls by FRP structural repointing. Proc., 5th Int.Conf. on Fibre Reinforced Plastics for Reinforced Concrete Structures, Thomas Telford, Cambridge, U.K., 1033–1042.

TNO Report (2004). Shear test on masonry panels; literature survey and proposal for experiments. TNO Building and Construction Research 2600 AA Delft.

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IMPORTANCE OF HYSTERETIC CONSTITUTIVE MODEL FOR HIGH-STRENGTH REINFORCING STEEL IN SEISMIC ANALYSIS OF REINFORCED CONCRETE (RC) STRUCTURES

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ABSTRACT

The hysteretic behaviour of reinforced concrete structures depends to a large extent on the hysteretic behaviour of reinforcing steel. An accurate and computationally efficient numerical model of reinforcing steel is, thus, very important in the analysis and evaluation of these structures under cyclic loads, including earthquake loads which is also cyclic in nature. Despite the potential benefits of higher strength reinforcement, there are still a number of important criteria for seismic concern like hysteretic energy dissipation and drift need to be solved which are not possible to know completely from monotonic analysis. A brief review of nonlinear hysteretic constitutive model for ordinary and high-strength steel is discussed in this study with their advantages and disadvantages along with few cyclic tests of RC members made of normal and high-strength steel.

Keywords: Hysteretic, reinforced concrete, high-strength steel,seismic, nonlinear,cyclic tests

1. INTRODUCTION

Although, the hysteretic behavior of reinforced concrete structures depends on hysteretic behavior of reinforcing steel, concrete and the bond-slip properties, but for flexural dominated RC structures it depends to a large extent on the hysteretic behavior of reinforcing steel and in this paper only the influence of cyclic constitutive characteristics of reinforcing steel on the behavior of RC structures is discussed. An accurate and computationally efficient numerical model of reinforcing steel is, thus, very important in the analysis and evaluation of these structures under cyclic loads, including earthquake loads.

Some important criteria for seismic concern like energy dissipation and drift are not possible to know completely from monotonic analysis and there is a clear difference between the specimens reinforced with traditional steel and those reinforced with high-strength steel is the hysteretic energy dissipation. Because the specimens reinforced with high-strength steel are less stiff, the area contained inside the hysteretic loops is smaller. Therefore, they will dissipate less energy due to hysteresis. All parameters used for generating the hysteretic model for ordinary and high-strength reinforcing steel are not same as stated in Balan et al. (1998). Although the advantages and disadvantages of grade 75 steel over grade 60 in Reinforced Concrete are described in details in Noor and Ahmed (2008); Noor (2010) under monotonic load, it is required to get the real scenario under cyclic load since the nature of seismic action on structures is cyclic.

There is growing interest within the reinforced concrete industry in using higher strength reinforcing steel for certain applications. This interest is driven primarily by relief of congestion; particularly in buildings assigned a high seismic design category. There are also other areas where high strength bar can help improve construction efficiencies, or—combined with high-strength concrete—allow reinforced concrete to be used in more demanding applications. Today, the vast majority of concrete design and construction uses Grade 60 steel, with occasional but increasing use of Grade 75. The benefits of increased strength, smaller dimensions and lower volumes would see its immediate application into design. In the last few years, a draft standard incorporating the use of high strength 500 MPa (72.50 ksi) steel to the construction industry was introduced.

Despite the potential benefits of higher strength reinforcement, there are still a number of important questions that have to be answered from design and production to fabrication and placement. It is found that high strength steel has inadequate ductility compared with ordinary strength reinforcing steel. Before using high strength steel in RC members in seismic prone areas, it is necessary to do cyclic experimental test or modelling RC structures using appropriate cyclic constitutive model for the specific steel to be used in the RC structures. A brief review of hysteretic constitutive model for ordinary and high-strength steel is discussed in this study with their advantages and disadvantages along with few cyclic tests of RC members made of normal and high-strength steel available in the literature and at the end it is explored that it is necessary to know the appropriate cyclic characteristics of high-strength steel either through tests or modelling in order to get real response of seismic analysis of RC structures.

2. A REVIEW OF CONSTITUTIVE MODEL FOR ORDINARY AND HIGH-STRENGTH STEEL UNDER CYCLIC LOADING

An important issue in the research for an appropriate reinforcing steel model is its numerical efficiency. A large number of iterations, and consequently evaluation of any steel stress-strain function, is performed at each load step. This necessitates the use of a simple material model. Counteracting the need for simplicity is the requirement for accuracy, which is based on the observations that cracks running through the depth of the member can remain open during moment reversals, causing the hysteretic response of the cross-section to be solely controlled by the behaviour of reinforcing steel. The behaviour of reinforcing steel may control the response of reinforced concrete structural elements subjected to earthquake loading. Thus, it is necessary to develop an analytical model that predicts the fundamental characteristics of steel within a range of loading that is appropriate for these structural systems. Typical load histories for reinforcing steel follow from consideration of the observed response of reinforced concrete structural elements. Characteristics of steel response are established through laboratory testing of steel coupons. Experimental data provide information for final calibration and refinement of the proposed model.

The available models for stress-strain relationship for cyclic behaviour can be classified into two groups. In the first the history dependence in the material behaviour is considered. This group is referred to as history dependent formulations. The other is based on generalizations of Ramberg-Osgood equations. In this approach, history dependence is also taken into account, but not in a direct manner. Model by Menegotto and Pinto (1973) or the one by Ma et al. (1976) are two well-known models of the second category. Only Ramberg-Osgood models (1943) are implemented in the SeismoStruct with the first proposed bilinear kinematic hardening model. Monti-Nuti (1991) model accounts for inelastic buckling of reinforcing bars. The Menegotto-Pinto model with Filippou isotropic hardening rules is discussed below.

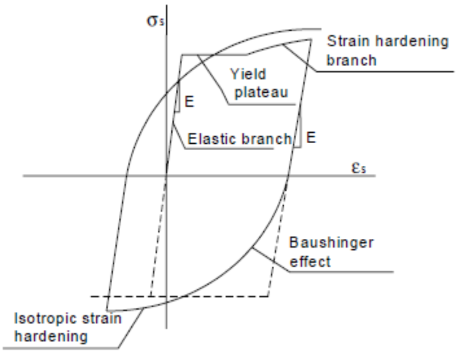

Figure 1: Main characteristics of stress-strain relationship of reinforcing bars under cyclic loading

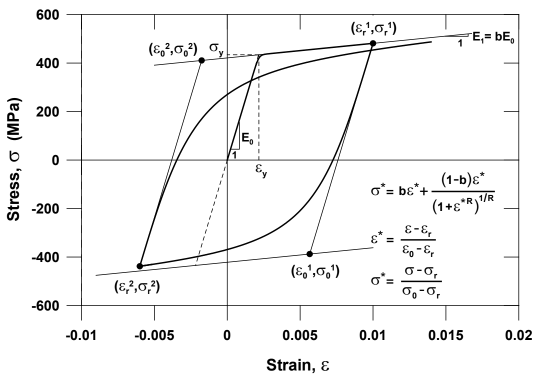

Figure 2: Constitutive Model for Steel

Figure 1 shows the main characteristics of the hysteretic constitutive model for reinforcing steel. The well-known nonlinear hysteretic constitutive model proposed by Menegotto and Pinto (1973) and extended by Filippou et al. (1983) for reinforcing steel is popular now a day as shown in Figure 2. This constitutive model considers Bauschinger's effect and the influence of both kinematic and isotropic strain hardening on the hysteretic uniaxial stress – strain behavior of reinforcing steel bars. The hysteretic model accounts for the degradation of strength properties with accumulation of plastic strains. The material parameters of the model are calibrated with monotonic tests of coupon specimens, while strength degradation relations are derived from cyclic test data. Correlation studies of the model with available experimental data for ordinary and high-strength reinforcing steel demonstrate the ability of the model to simulate the hysteretic behaviour of all types of reinforcing steel over a wide range of strain variations.

3. EXPERIMENTAL AND ANALYTICAL TESTS OF RC MEMBERS REINFORCED WITH NORMAL AND HIGH-STRENGTH STEEL

Rautenberg et al. (2010) conducted tests of columns reinforced with normal and high-strength reinforcements under cyclic load reversal. The focus of his work was on columns meeting requirements for columns of special moment frames and columns with axial loads not exceeding the axial load at balance. The flexural strength of these columns was controlled by the strength of

the steel following two sections with different grades of steel having similar moment capacities as long as the product of reinforcement ratio and yield stress was similar for both sections. Experimental and analytical tests of this hypothesis were presented. Tests of columns reinforced with either 60-ksi or 120-ksi steel were conducted. According to them, although the moment-curvature analysis provides insight, it is difficult to translate sectional behaviour into the global behaviour of a column. Therefore, a series of tests were conducted on columns reinforced with high-strength steel. A matrix of the test program can be seen in Table 1 and a drawing of the specimens to be tested can be seen in Figure 3. Details are described in Rautenberg et al. (2010).

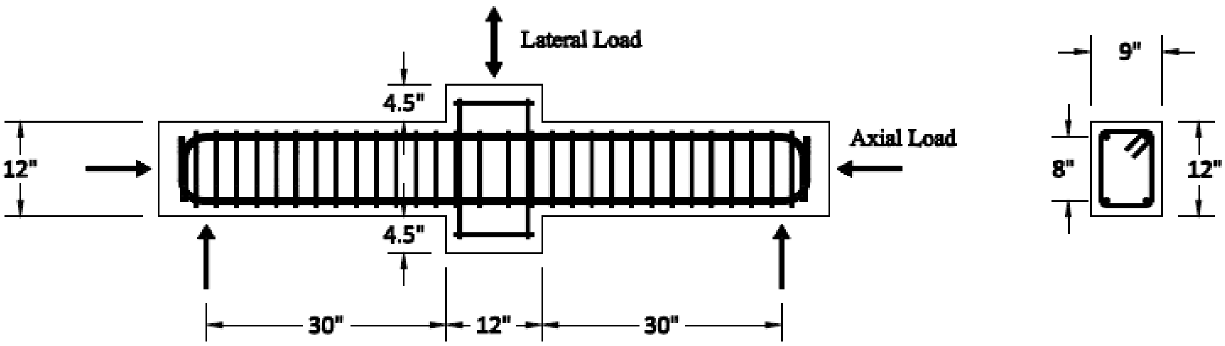

Figure 3: Drawing of typical test specimen

Table 1: Test matrix of relevant specimens

Specimen No.	***Fy*(ksi)**	***As / Ag***	***P / (f'c Ag)***
1	60	3.3	0.10
2	120	1.6	0.10
8	60	2.4	0.20
9	120	1.1	0.20

Where: *fy* = yield stress of longitudinal reinforcement,
As = total cross-sectional area of longitudinal reinforcement,
Ag = gross cross-sectional area of column,
P = applied axial load, and
f'c = compressive strength of concrete.

The two important comparisons to make in this series of tests were between specimens 1 and 2, and between specimens 8 and 9. In these two sets, all variables were held constant except for the strength and amount of reinforcement provided, and the applied axial load. The longitudinal reinforcement in the column pairs was proportioned such that the moment capacities would be nearly equal. Specifically, specimens 2 and 9 had about half the amount of steel as specimens 1 and 8, respectively. The other variable considered was the applied axial load. In specimens 1 and 2, the applied axial load was 0.10 *f'c Ag,* while it was twice that in specimens 8 and 9. The sizes of longitudinal reinforcing bars ranged from #5 to #7 in the four columns described. For all columns, shear and confinement reinforcement was provided by #3 bars spaced at 2.5 inches. The shear-drift curves from the four tests described can be seen in Figures 4 through 7.

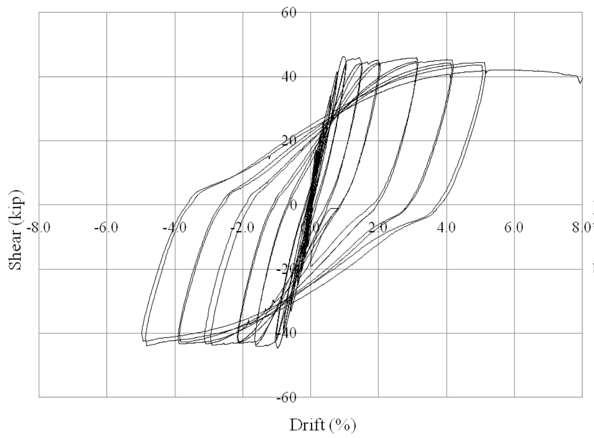

Figure 4: Shear-drift plot for specimen 1

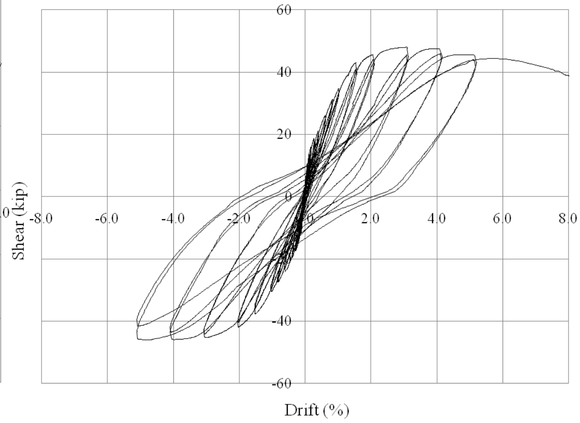

Figure 5: Shear-drift plot for specimen 2

First, a comparison between specimens 1 and 2 was made as shown in Figures 4 and 5. As was expected, the moment capacity of both columns was approximately the same even though the column reinforced with high-strength reinforcement had about half the amount of steel. Both columns exhibited large deformation capacities – exceeding 8% drift ratio. However, at the end of the response, it can be seen that specimen 2 was losing resistance at a faster rate than specimen 1.

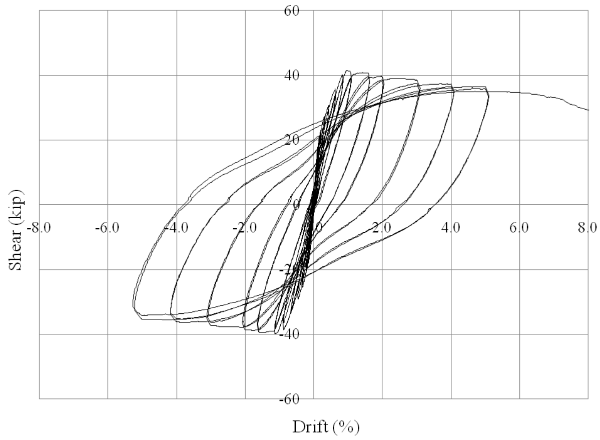

Figure 6: Shear-drift plot for specimen 8

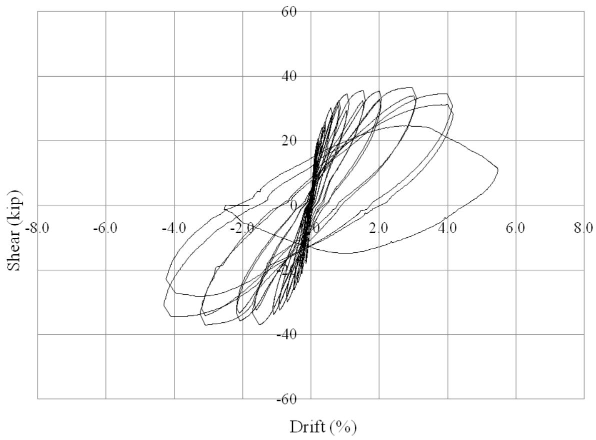

Figure 7: Shear-drift plot for specimen 9

Some of the same trends seen in the comparison between specimens 1 and 2 can be seen when comparing specimens 8 and 9 as shown in Figures 6 and 8. In this comparison, the trends became more apparent. This is because the axial load was twice as high, which, in a way, amplifies the negative aspects of both columns. The moment capacities were nearly the same in the two specimens, except the one using high-strength reinforcement, specimen 9, had a moment capacity about 10% lower than specimen 8. The difference in strength of the columns was nearly proportional to the difference in the product of the reinforcement ratio and the yield stress of the steel. Specimen 9 (high-strength reinforcement) failed at a drift ratio of about 4% while specimen 8 (traditional reinforcement) failed at a drift ratio exceeding 7%. In both cases, the failure mode was buckling of longitudinal reinforcement, even though the transverse hoops were spaced at a quarter of the effective depth of the column (2.5 inches). The difference, however, is the method in which the bars buckled.

Another clear difference between the specimens reinforced with traditional steel and those reinforced with high-strength steel was the hysteretic energy dissipation. The cause of this was the stiffness of the columns. Because about half the steel is used in specimens 2 and 9, the stiffnesses of specimens 2 and 9 were about half those of specimens 1 and 9, respectively. Because the specimens reinforced with high-strength steel were less stiff, the area contained inside the hysteretic loops was smaller. Therefore, they will dissipate less energy due to hysteresis.

4. CONCLUSIONS

In predicting the response of reinforced concrete members under earthquake induced forces, it is necessary to estimate the cyclic stress-strain behavior of the reinforcing steel. For the purpose of inelastic analysis, appropriate steel models are necessary. The model employed to represent the behavior of the steel rebar determines to a large extent the accuracy of the analysis. Despite the potential benefits of higher strength reinforcement, there are still a number of important criteria for seismic concern like hysteretic energy dissipation and drift need to be solved which are not possible to know completely from monotonic analysis. Because there is a clear difference between the specimens reinforced with traditional steel and those reinforced with high-strength steel is the hysteretic energy dissipation. Although the moment-curvature analysis provides insight, it is difficult to translate sectional behavior into the global behavior of a RC column. Therefore, a series of tests need to be conducted on columns reinforced with high-strength steel especially when the behavior of these columns is subjected to cyclic displacement reversals. Therefore in order to get real response of seismic analysis of RC structures reinforced with high-strength steel, it is necessary to implement appropriate steel model with cyclic constitutive characteristics in the RC model and cyclic test of RC members to know the hysteretic energy dissipation and drift.

REFERENCES

Balan, T.A., Filippou, F.C. and Popov, E.O. (1998), "Hysteretic model of ordinary and high-strength reinforcing steel"., Journal of Structural Engineering 1998;124(3):288-297.

Filippou, F.C., Popov, E.G. and Bertero. V.V.(1983), "Effects of bond deterioration on hysteretic behaviour of reinforced concrete joints", EERC report no. UCB/EERC-83/19, Earthquake Engineering Research Center, University of California, Berkeley, CA, USA.

Menegotto, M. and Pinto, E.(1973), "Method of analysis for cyclically loaded reinforced concrete plane frames including changes in geometry and non- elastic behavior of elements under combined normal force and bending". IABSE symposium on resistance and ultimate deformability of structures acted on by well-defined repeated loads, Lisbon, Portugal.

Ma SYM, Bertero, V. V. and Popov E. P.(1976), "Experimental and analytical studies on the hysteretic behaviour of reinforced concrete rectangular and T-beams", EERC Rep. 76-02. Earthquake Engrg. Res. Ctr., University of California. Berkeley, Cali.

Monti, G. and Nuti, C.(1991), "Analytical model of cyclic behaviour of reinforcing bars with inelastic buckling", Individual Report, Dept. of Struct. Engrg., Rome.

Noor, M.A. and Ahmed, A.U.(2008), " Study on Grade 75 and 60 Reinforcement in RC Design", BSRM Seminar, 12 April, 2008.

Noor, M.A.(2010), "Design with Grade 500 Steel", The University Press Limited, 61 Motihjeel, Dhaka, Bangladesh.

Ramberg, W. and Osgood, W. R.(1943), "Description of stress-strain curves by three parameters", NACA Rep. TN-902,. National Advisory Committee on Aeronautics, Washington. D.C.

Rautenberg, J.M., Pujol, S., Tavallali, H. And Lepage, A.(2010), "Cyclic Response of Concrete Columns Reinforced with High-Strength Steel", Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering, Toronto, Ontario, Canada.

DETERMINATION OF COMPRESSIVE STRENGTH OF MORTAR DUE TO FILLER EFFECT OF POZZOLANS

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ABSTRACT

Pozzzolans contribute in two fold of effects in concrete or mortar; i.e. physical or filler effect and chemical or pozzolanic effect. The filler effect is defined as proper arrangement of small particles into the microstructure that fill the voids and contribute towards improvement of compressive strength without any chemical reaction. Filler effect of pozzolan plays a vital role for the production of high strength mortar. This effect is dominating when pozzolan particles are in chemically inactive form. The individual contribution of physical and chemical effect in concrete and mortar still not determined. Several studies have been found in the published literature on finding the filler effect of pozzolans by replacing cement with chemically inactive materials which size is same as pozzolans. The chemically inactive materials used in previous studies are carbon black, limestone filler and ground river sand, which used bigger range size of replacement percentages (like 5%, 10%, 15% or 10%, 20%, 30%etc). However in this study, lower range size of replacement percentages (like 2.5%, 5%, 7.5% etc) were examined. This is due to probabilities of peak value of compressive strength due to physical effect may lie in between two replacement percentages used in previous studies. In order to determine the filler effect, chemically inactive material (ground river sand) with various particle sizes used as supplementary material of cement to produce mortar specimens. Result shows that compressive strength of ground river sand mortar at smaller replacement percentages is very near to the compressive strength of control mortar. The loss of compressive strength indicates the only filler effect of small size ground sand whereas pozzolanic effect was inactive in the concrete microstructure.

Keywords: voids, physical effect, replacement percentages, ground river sand, compressive strength

1. INTRODUCTION

Nowadays, many types of pozzolans are used globally such as Rice Husk Ash (RHA), Fly Ash (FA), Palm Oil Fuel Ash (POFA), Olive Oil Ash (OOA), Sugar Cane Baggasse Ash (SCBA) etc. After proper incineration and ground [Khan et. al. 2013] these pozzzolans mainly used as a partial replacement for Portland cement in paste, mortar and concrete. Because it is seen that the compressive strength of control mortar and concrete is lower than the compressive strength of concrete and mortar which is partially replaced by pozzolans. Actually, pozzolans when present in mortar or concrete, contributes to the total compressive strength of mortar or concrete in two ways: by the physical or filler effect and by the or chemical pozzolanic reaction named as chemical effect of pozzolans [Jaturapitakkul *et al*., 2011].The concomitant action of both physical and chemical effect gives higher compressive strength of mortar and concrete. But it is essential to know the exact contribution of each effect. Normally filler effect is defined as the packing charateistics of the mixture, which depend on size, shape and texture of the particles and also chemically inactive [Cordeiro *et al*., 2008]. Whereas chemical effect is occurred due to chemical reaction between cement hydration product ($Ca(OH)_2$) and active silica (SiO_2) present in pozzolans.(Jamil *et al*., 2013). Determining how much of the compressive strength is due to the filler effect or the pozzolanic reaction is still unknown to the researchers. Because ASTM C618 does not separate the physical effect from chemical effect of pozzolans. As a result to distinguish these two effects different researchers were followed different methods.

Detwiler and Mehta (1989) and Goldman and Bentur (1994) used carbon black to check microfiller effect or packing effect through the cementitious system. They found that a large amount of strength increased due to filler effect only. According to Isaia et al (2003) was used limestone filler (chemically inactive) and reported that filler effect greater than pozzolanic effect. Recently researchers preferred to use ground river sand as partial

replacement of cement to verify filler effect. This due to the availability of river sand as well as chemically inactive nature. According to Jaturapitakkul et. al.(2011) was examined the filler effect using ground river sand with three different particle sizes up to 40% replacement of ordinary Portland cement. Filler effect was found clearly when used small ground sand. Most of the researchers used bigger range of percentages replacement (such as 5%, 10%, 15% or 10%, 20%, 30% etc) of cement to verify filler effect. But they found less compressive strength when used insoluble material compared to control mortar. It is important to know how much effect will occur when using lower range lower range of percentages replacement (such as 2.5%, 5%, 7.5% etc), because the probabilities of peak value of compressive strength (i.e. greater compressive strength of insoluble material mortar compared to the control mortar) due to physical effect may lie in between two replacement percentages used in previous studies.
This paper describes a method to determine compressive strength of mortar due to filler effect of pozzolans. In this study only used small sized ground sand with lower range of percentages replacement of cement up to 20%.

2. EXPERIMENTAL PROGRAM

2.1 Materials

Materials used in this investigation consisted of Portland cement type I, Standard sand as required by BS EN 196-1 (2005) as fine aggregate. To prepare non-reactive material, natural sand found in malaysia was washed by water and sun-dried for 2–3 days to reduce its moisture content to be less than 0.1%.Then, it was ground by ball mill to reduce its size as much as possible.
Small ground sand (SGS) particles

About 5 ± 2% by weight of the materials were retained on a 45-µm sieve. This size was selected for this study

2.2 Detail of mortars and test of specimens

A constant ratio of cementitious materials (Portland cement type I plus small ground sand) to standard sand was set at 1 to 2.75 by weight, and water to cementitious materials ratio was maintained at 0.485. The mix proportions of mortar containing small ground sand are shown in Table 1. Portland cement type I was replaced by ground sand at the rate of 20% by weight of cementitious materials. All mortars were casting 50*50*50-mm standard molds and removed from the molds after casting for 24 h are shown in Figure 1

Table 1: Mix proportions of mortar containing small ground sand

Specimen	Cement	Fine aggregate	SGS	W/B ratio
Control	1	2.75	0	0.485
SGS2.5	0.975	2.75	2.5	0.485
SGS5	0.95	2.75	5	0.485
SGS7.5	0.925	2.75	7.5	0.485
SGS10	0.9	2.75	10	0.485
SGS12.5	0.875	2.75	12.5	0.485
SGS15	0.85	2.75	15	0.485
SGS17.5	0.825	2.75	17.5	0.50
SGS20	0.8	2.75	20	0.49

Figure 1 : Prepared small ground sand mortar specimens.

3. CHARACTERIZATION OF MATERIALS

3.1 Chemical composition analysis

The chemical composition of ordinary Portland cement and small sized ground sand was determined by using X-ray fluorescence (XRF) technique are shown in Table 2. For small ground sand the major constituent SiO2 with a concentration of 85.72% and the Al_2O_3 concentration was 3.73%. Other constituents, such as CaO, MgO, SO3, were less than 1%.

Table 2 Chemical composition of materials (in mass)

Compounds (%)	Small ground sand	Ordinary Portland Cement
SiO_2	**85.72**	**17.78**
Al_2O_3	**3.73**	**3.89**
Fe_2O_3	**1.08**	**2.96**
CaO	**0.32**	**62.69**
MgO	**0.09**	**2.32**
Na_2O	**0.09**	**---**
K_2O	**0.85**	**0.32**
SO_3	**0.04**	**4.11**
P_2O_5	**0.04**	**0.05**
MnO	**0.01**	**0.08**

3.2 X-ray diffraction analysis

The X-ray diffraction analyses were performed to identify differences in the formation of crystalline or amorphous silica. The very sharp peak for ground sand are shown in Figure 2 indicates the crystalline nature of silica, whereas amorphous form due to the broad peak on 2θ angle of 22° [Nair D.G. et.al. 2008].

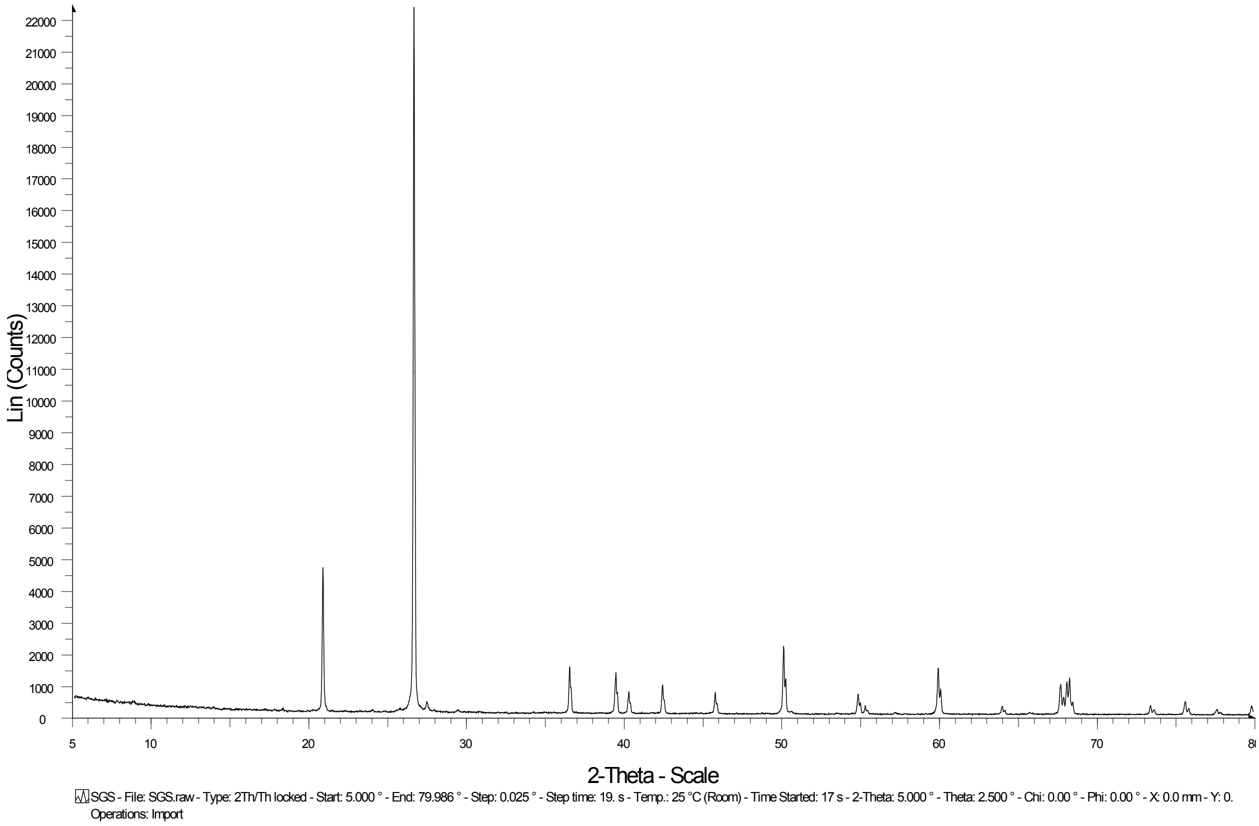

Figure 2 X-ray spectrum for small ground sand

4. RESULTS AND DISCUSSION

The compressive strengths of the ground sand mortars with a cement replacement proportion of 2.5%, 5%, 7.5%, 10%, 12.5%, 15%, 17.5% and 20% at 7 days are shown in Table 3.The mortar containing small ground sand showed lower compressive strength than control mortar at 7 days curing. The difference of compressive strength between control mortar and ground sand mortar is gradually increasing with the percentages replacement. This is due to the chemically inactive nature of ground sand and it only took part in filler effect into the microstructure. As a results at lower replacement percentages the compressive strength of ground sand very near to the control mortar and it was only 0.11 shown in Figure 4 when 2.5 % cement replaced by ground sand. But when cement replaced by pozzolans extra compressive strength found due to pozzolanic reaction.

Table 3: Compressive strength of small ground sand mortar

Specimen	Compressive strength (MPa)
	7 days
Control	32.91
SGS2.5	32.80
SGS5	32.58
SGS7.5	31.20
SGS10	31.00
SGS12.5	30.50
SGS15	30.00
SGS17.5	29.14
SGS20	28.50

Compressive strength of mortar was determined using 50 mm cube specimens based on ASTM C109 (2009) testing standard. Compressive strength test was performed with maintaining a loading rate of 1600N/sec (900-1800 N/sec) according to ASTM C109 (2009) testing standard shown in Figure3.

Figure 3 Picture of the compression machine used in determine compressive strength test of cube mortar specimens.

Figure 4 shows the loss of compressive strength of small ground sand mortar compared to the control mortar at 7 days curing in different percentages replacement rate.It is seen that the relation between these two were linearly increasing. Moreover, the figure also suggested that no or little compressive strength of the ground sand mortar was contributed from the pozzolanic reaction and filler effect only reponsible for this strength.

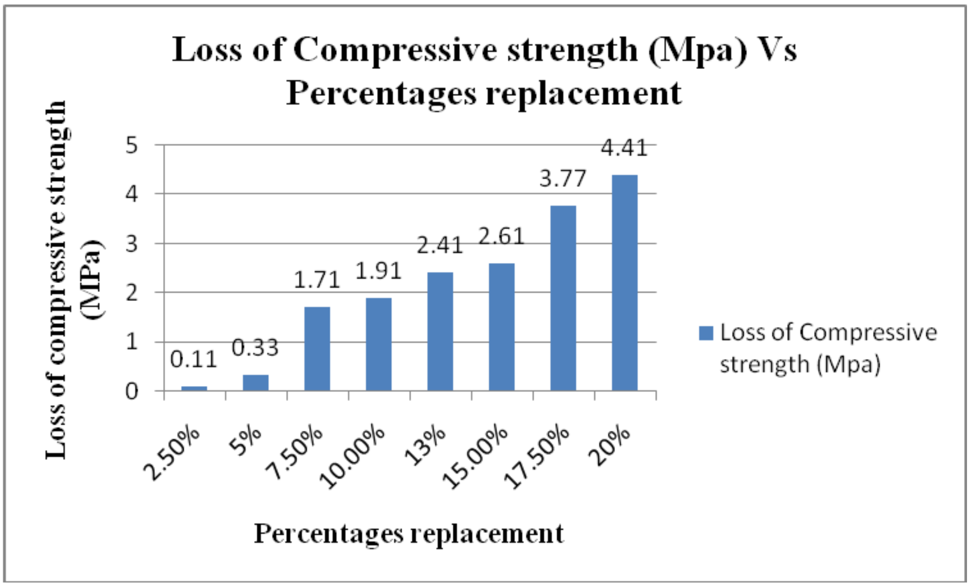

Figure 4 Relation between percentages replacement and compressive strength of mortar

5. CONCLUSION

This study aimed to determine compressive strength due to filler effect of pozzolans through a experimental program. The following observations can be concluded as:

1. When using small ground river sand the compressive strength of ground sand always lower than the control mortar, this is due to absence of any kind of chemical reaction between cement and ground sand mortar.
2. But at 2.5% replacement, the difference of compressive strength between control mortar and ground sand mortar very low because of filler effect of small ground sand mortar.
3. It is also seen that at 20% replacement gives significant compressive strength, so it is clear that the filler effect important as much as pozzolanic effect.

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REFERENCES

ASTM C109. 2009. Standard test methods for compressive strength of hydraulic cement using 50mm cube specimens, ASTM C109-09, Annual book of ASTM standard 04.01, pp. 78-85.

ASTM C618. 2001. Standard specification for coal fly ash and raw or calcined natural pozzolan for use as a mineral admixture in concrete, ASTM C618-00, Annual book of ASTM standard 04.02. pp. 310-313.

BS EN 196-1. 2005. Methods of testing cement, Part 1: Determination of strength. pp.1-33.

Cordeiro GC, Toledo Filho R.D. 2008. Tavares LM, Fairbairn EMR. Pozzolanic activityand fillers effect of sugar cane bagasse ash on Portland cement and lime mortars. Cement and Concrete Composites. 30(5):410–8

Detwiler, R.J. and Mehta. P.K. 1989. Chemical and Physical Effects of Silica Fume on the Mechanical Behavior of Concrete. ACl Materials Journal. 86:609-614.

Goldman A, Bentur A. 1994. Properties of cementitious systems containing ilica fume or nonreactive micro fillers. Advanced Cement Based Materials. 1(5):209–15

Isaia GC, Gastaldini ALG, Morases R. 2003. Physical and pozzolanic action of mineral additions on the mechanical strength of high performance concrete. Cement and Concrete Composites. 25(1):69–76.

Jamil, M., Kaish, A.B.M.A., Raman, S.N., Zain, M.F.M. 2013. Pozzolanic contribution of rice husk ash in cementitious system. Construction and Building Materials. 47:588-593.

Jaturapitakkul C, Tangpagasit J, Songmue S, Kiattikomol K. 2011. Filler effect and pozzolanic reaction of ground palm oil fuel ash. Construction and Building Materials. 25(11):4287–4293.

M. N. N. Khan, Jamil, M., Kaish A.B.M.A., Zain , M.F.M., 2013.Production of Rice Husk Ash And Its Effect on Cementitious System: A Review. International Conference on Engineering and Built Environment (ICEBE),UKM,Selangor Darul Ehsan,Malaysia.

Nair DG, Fraaij A, Klaassen AAK, Kentgens APM. A structural investigation relating to the pozzolanic activity of rice husk ashes. Cement and Concrete Research. 38(6):861–9.

Cheerarot, R. , Tangpagasit, J., Jaturapitakkul, C. 2004. Compressive Strength of Mortars Due to Pozzolanic Reaction of Fly Ash. Special Publication. 221, 411-426

Tangpagasit, J., Cheerarot, R., Jaturapitakkul, C., Kiattikomol, K. 2005. Packing effect and pozzolanic reaction of fly ash in mortar. Cement and Concrete Research. 35(6):1145–1151.

PERFORMANCE OF RECYCLED WASTE CONCRETE AND ITS APPLICABILITY IN CONSTRUCTION INDUSTRY

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ABSTRACT

Urbanization demands the renovation of the old structures commonly in the developing countries, resulting in a huge amount of construction debris annually. The construction debris due to failure of aged or faulty structures is considered as waste concrete which are normally composed of concrete rubble, bricks and tiles, sand and dust, timber, plastics, cardboard and paper, and metals. Recycled waste concrete is manufactured from waste concrete by recycling the aggregates of waste concrete. As well as the demand for natural coarse aggregate is also increasing, the scarcity of it may happen. So, proper recycling of waste concrete is important to have an alternative material for utilization in concrete and to reduce the high consumption of natural aggregate. To achieve the suitability of recycled waste concrete the strength and durability of it are confirmed first. In this work, five type of sample are made by varying the percentage of recycled and fresh coarse aggregates(brick aggregates and stone chips) in which workability is constant. Compressive and tensile strength decreases in accordance with increasing amount of recycled coarse aggregate at 7, 28 and 90 days. RCPT (Rapid Chloride Permeability Test) is performed on hardened concrete.

Keywords: Waste concrete, recycling, mechanical properties, permeability, usability.

1. INTRODUCTION

Concrete is a mixture of cementious material, aggregate, and water. The cementious material known as binding material, binds the individual units of aggregates unto a solid mass with the help of water. Aggregate is commonly considered as inert filler, which accounts for 60 to 80 percent of the volume and 70 to 85 percent of the weight of concrete. Although aggregate is considered inert filler, it is a necessary component that defines the thermal and elastic properties and dimensional stability of concrete. Aggregate is classified as two different types, coarse and fine. Concrete is a widely used construction material, so its mechanical and other properties are very important factor. Concrete has relatively high compressive strength, but lower tensile strength, which ranges from about 7 to 10% of the compressive strength. Though strength gives the overall view of quality of concrete but in practical case, durability, impermeability, workability may take into account.

Concrete is the premier construction material across the world and the most widely used in all types of civil engineering works. As urbanization and modernization accelerate the prosperity of a country so, during the past years, in most developing countries, a large number of old constructions have been demolished and millions of tons of construction debris have been produced. For example, two billion tons of aggregate are produced each year in the United States. Production is expected to increase to more than 2.5 billion tons per year by the year 2020 (M. Malesev et al. 2010). This situation leads to a question about the preservation of natural aggregates sources. Through proper recycling process the waste concrete can be used in new concrete production. The possibility of recycling waste concrete from the construction industry is thus of increasing importance (Robinson et al.2004). So, it helps to meet the demand for natural coarse aggregate. Toward this goal, diverse policies and exploitation programs have been proposed by the academia and industry (Padmini et al.2002; Limbachiya et al.2000). Building rubble could be transformed into useful recycled aggregates through proper processing (Chen et al. 2002). The recycled aggregates chosen by the writer contained bricks and tiles. The test results showed that the compressive strength of concrete made with recycled coarse aggregate containing brick and tile particles is about 75–80% that of normal concrete. Using unwashed recycled aggregate in concrete will affect its strength. Poon et al. (2002) in their study used recycled aggregates from C and D (construction and demolition) wastes sourced from two public filling areas. The mixes were varied by replacement of natural coarse and fine aggregate with recycled aggregates up to 100% by weight, with or without the incorporation of fly ash. The test results conducted by the writer showed that the replacement of coarse and fine natural aggregates with recycled aggregates at the levels of 25% and 50% had little effect on the compressive strength of the bricks and blocks, but higher levels of replacement reduced the compressive strength.

2. OBJECTIVE

The aim of the study is to determine the performance of recycled waste concrete and its applicability in durable concrete production. In pursuit of this aim, the following objectives have been set:

- To measure the strength property of concrete manufactured with recycled concrete aggregate.
- To observe the permeability of waste concrete.
- To investigate the applicability of recycled waste concrete.

3. METHODOLOGY

To fulfill the objectives of this work, the following steps should be performed:

- Collection of material.
- Crushing and recycling the material.
- Material properties.
- Preparation of specimen according to ASTM C 39-93a.
- Curing of specimen.
- Compressive strength test and splitting tensile strength test according to ASTM C 39 and ASTM C 496 respectively.
- Rapid Chloride Permeability Test according to ASTM C 1202
- Ascertaining the charge passed through the samples.
- Analyzing test results and plotting relevant graph.
- Conclusion.

Table 3.1: Mixing percentages of fresh and recycled brick aggregates and stone chips

Sample	A	B	C	D	E
Cement: FA: CA	1:1.8:3	1:1.8:3	1:1.8:3	1:1.8:3	1:1.8:3
Percentages of Fresh aggregate	100	67	33	0	100
Percentages of Recycled aggregate	0	33	67	100	0
Admixture/Cement ratio	0	0	0	0	400/1000

(gm/gm)						
Slump value (inch)	Brick aggregates	3	3	3	3	3
	Stone chips	3	3	3	3	3

N.B.[FA= Fine Aggregates, CA= Coarse Aggregates]

4. ILLUSTRATIONS

4.1 Compressive strength Test

According to the ASTM C 39, compressive strength test was performed in the laboratory by compressive strength testing machine.The specimens size were Φ4 inch x L8 inch .

Sample A is considered as controlled sample. Change of strength from the controlled samples is given in following bar diagram. Sample E has higher compressive strength at any days.

4.1.1 Using Brick Aggregates

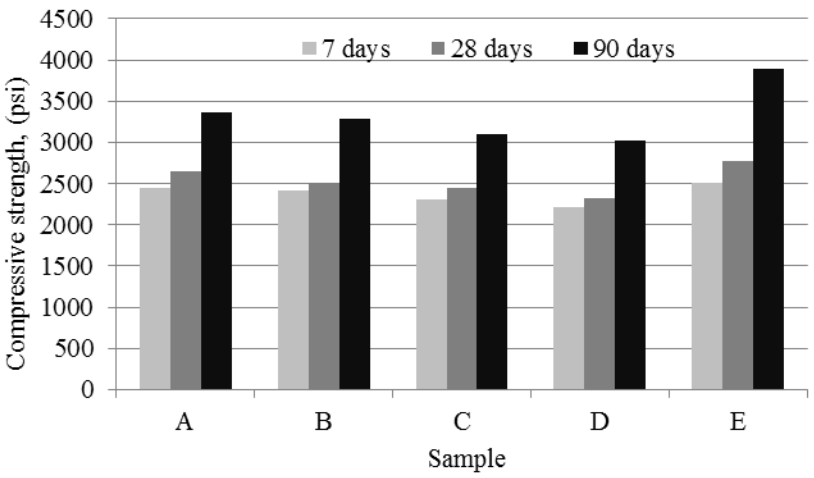

Figure 4.1.1: Bar diagram of compressive strength for five samples (brick aggregates)

The 90 days compressive strengths of sample B, C and D were respectively 2.38%, 7.74% and 10.12% less than that of controlled sample A (Figure 4.1.1). So it can be said that 33% replacement of fresh aggregate by recycled aggregate causes no remarkable deviation in compressive strength. It is also shown that up to 100% replacement of fresh aggregate by recycled aggregate, the obtained strength value is higher than the required minimum value for ordinary construction works. The compressive strength of sample E was15.8% higher than that of controlled sample A.

4.1.2 Using Stone Chips

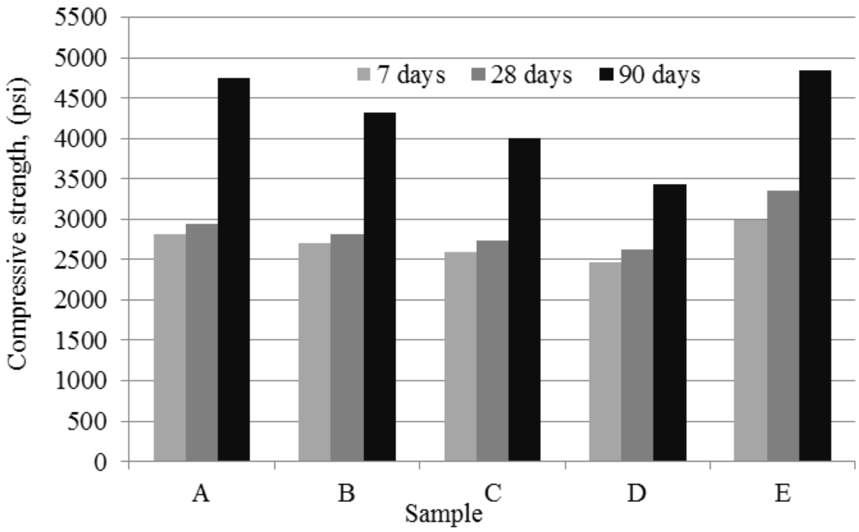

Figure 4.1.2: Bar diagram of compressive strength for five samples (stone chips)

The 90 days compressive strengths of sample B, C and D were respectively 8.86%, 15.61% and 27.64% less than that of controlled sample A (Figure 4.1.2). So it can be said that 33% replacement of fresh aggregate by recycled aggregate causes no remarkable deviation in compressive strength. It is also shown that up to 100% replacement of fresh aggregate by recycled aggregate, the obtained strength value is higher than the required minimum value for ordinary construction works. The compressive strength of sample E was 2.3% higher than that of controlled sample A.

Stone chips has shown the better performance than brick aggregates in compressive strength test.

4.2 Splitting Tensile strength

According to the ASTM C 496, splitting tensile strength test was performed in the laboratory by compressive strength testing machine.The specimens size were Φ4 inch x L8 inch.

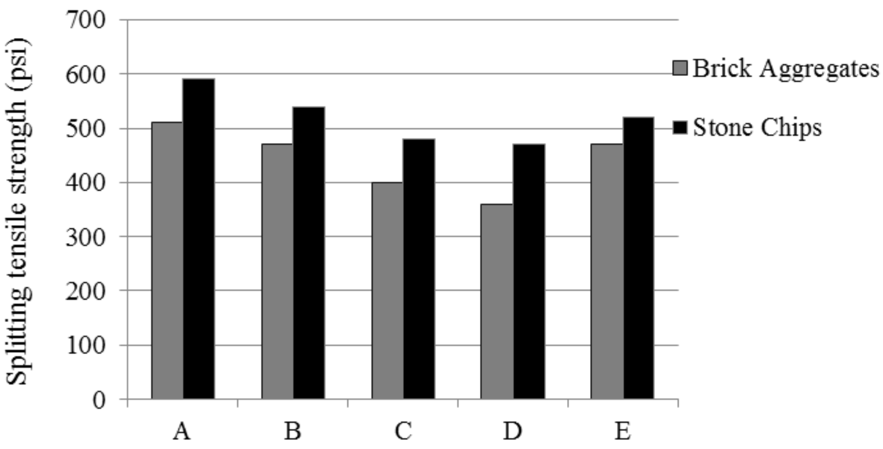

Figure 4.2.1: Variation of splitting tensile strength of five samples (brick aggregates and stone chips)

From Figure 4.2.1, it is shown that , in case of using brick aggregates the tensile strengths of sample B, C and D were respectively 7.84%, 21.57% and 29.41%,less than that of controlled sample A. Percentage replacement of fresh aggregate by recycled aggregate shows a uniform decreasing trend in tensile strength. Tensile strengths of sample E were 7.84% higher than that of controlled sample A.

In case of using stone chips, the tensile strengths of sample B, C and D were respectively 8.47%, 20.4% and 29.41%, less than that of controlled sample A. Percentage replacement of fresh aggregate by recycled aggregate shows a uniform decreasing trend in tensile strength. Tensile strengths of sample E were 11.86% higher than that of controlled sample A.

4.3 Permeability Result

According to ASTM C 1202, permeability test is performed. RCPT is done on hardened concrete specimen. In this test,charge is measured at 30 minutes interval for 6 hours.

4.3.1 Using Brick Aggregates

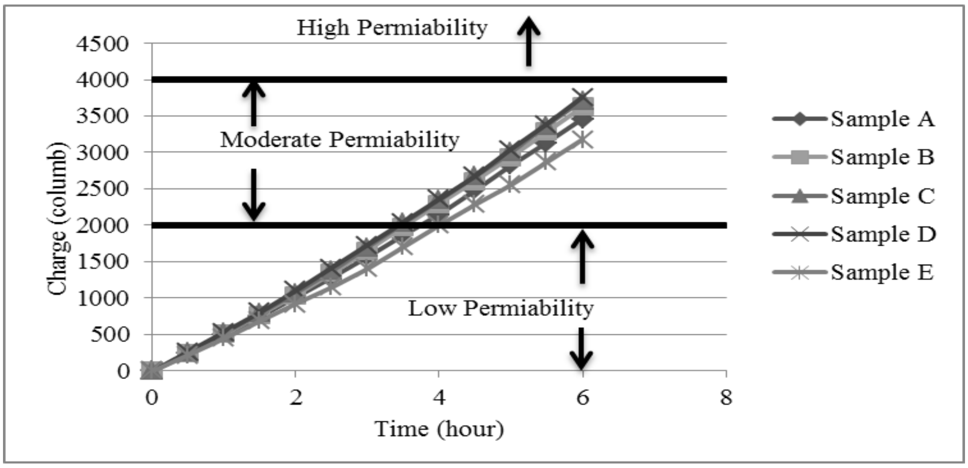

Figure 4.3.1: Comparison of charge passes through five samples (Brick Aggregates)

Figure 4.3.1 shows that, the concrete is moderate type for chloride ion permeability, which ranges between 2000-4000 coulombs and controlled sample shows less permeability than other samples having recycled aggregate. Moderate permeability allows the applicability in structural use, so 100% recycled waste concrete shows no remarkable effect on structure.

4.3.2 Using Stone Chips

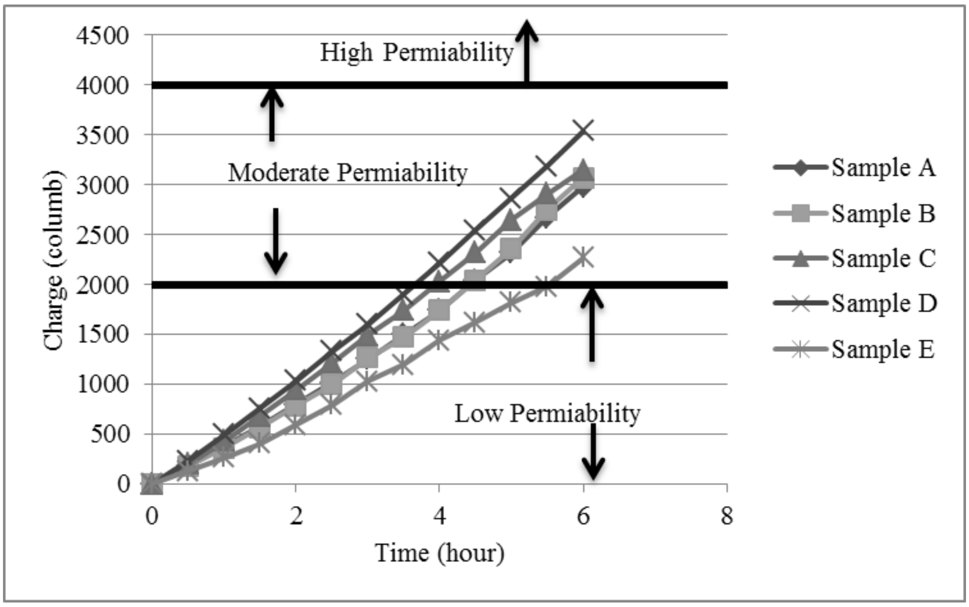

Figure 4.3.2: Comparison of charge passes through five samples (stone chips)

Figure 4.3.2, it is shown that the charge passing due to chloride ion ranges between 2000-4000 coulombs for all samples. Therefore, the concretes were moderate type for chloride ion permeability.

5. CONCLUSIONS

- In case of using brick aggregates, 100%, 67% and 33% replacement shows respectively 10.12%, 7.74% and 2.38% decreasing in compressive strength and 29.41%, 21.57% and 7.84% decreasing in tensile strength.
- In case of using stone chips, 100%, 67% and 33% replacement shows respectively 27.64%, 15.61% and 8.86% decreasing in compressive strength; 20.34%, 18.64% and 8.47% decreases in tensile strength.
- Chloride Ion Permeability ranges between 2000-4000 Coulombs for both cases (brick aggregates and stone chips).It indicates, all samples are categorized as moderate for chloride ion permeability. So, using 100% of recycled waste concrete has no remarkable effect on structure in case of permeability .
- Using admixture with recycled aggregate, shows better strength and permeability.
- In all tests stone chips shows the better performance with respect to brick aggregates.

From this test results, it may concluded that the applicability of recycled waste concrete is possible in concrete production.

6. RECOMMENDATIONS

Following recommendations may be considered for more betterment of the study:

- Effect of the mixing proportions of aggregates and binding materials should be studied.
- Alternative power supply system should be provided.
- The effect of admixture should be investigated broadly later.
- Financial analysis of recycled waste concrete should be done.

REFERENCES

C.S.Poon, S.C.Kou, L.Lam (2002) “Use of recycled aggregates in molded concrete brick and blocks”, Construction and Building Materials, Vol. 16, pp. 281–289.

How-Ji Chen, Tsong Yen, and Kuan-Hung Chen (2003). “Use of building rubble as recycled aggregates”, Cement and Concrete Research, Vol.33, pp. 125-132.

Limbachiya, M.C., Dhir, R.K., and Leelawat, T. (2000). “Use of recycled concrete aggregate in high-strength concrete”, Materials and Structure, Vol. 33(9), pp. 574–580.

Mirjana Malesev, Vlastimir Radonjanin and Snezana Marinkovic (2010). “ Recycled Concrete as Aggregate for Structural Concrete Production”, Sustainability, ISSN 2071-1050, Vol. 2, pp. 1204-1225.

Padmini, A. K., Ramamurthy, K., and Mathews, M.S. (2002). “Relative moisture movement through recycled aggregate concrete”, Magazine of Concrete Research, Vol.5, pp. 377–384.

Robinson Jr., G.R. Menzie, W.D., and Hyun, H. (2004). “Recycling of construction debris as aggregate in the mid-Atlantic Region, USA”, Resources Conservation and Recycling, Vol. 42(3), pp.275–294.

STUDY ON REINFORCEMENT CORROSION IN HIGH PERFORMANCE CONCRETE UNDER PRE-LOADING CONDITION

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ABSTRACT

The mutual effects between subjection of pre-loading and corrosion of reinforcing steel as well as their combined effect on serviceability were investigated. Three concrete beams of 1170 mm long x 100 mm wide x 150 mm high were used as test specimens. Two of them were made of high performance concrete incorporating fly ash and slag, while the other was control specimens of 100% OPC as binder. The beams were reinforced with a single Ø16-mm 60-grade bar and subjected to four-point loading at a constant load level of 80% of the average ultimate flexural strength. The corrosion process was accelerated by employing 5% NaCl solution. Galvanized current measurements were taken to monitor the corrosion initiation. After corrosion had initiated, an external current was applied to expedite the corrosion propagation, and responsive voltages were measured. Performance of the high performance concrete was evaluated in terms of corrosion resistance. The results suggested that for a rational service-life enhancement of reinforced concrete structures and effective corrosion resistance over a long-term period; influence of material propositions should be considered in combination with environmental aspects and service-load conditions.

Keywords: Corrosion;half-cell potential; supplementary cementitious materials; residual strength.

1. INTRODUCTION

Corrosion of reinforcement is one of the most common harmful damage that occurs in reinforced concrete. This damage reduces the service life of structural member and can create a safety hazard as the member is crushed. Steel in concrete is usually protected against corrosion by passivation of the steel arising from the high alkalinity of the pore solution with the concrete. A stable oxide layer is formed on the steel surface, which prevents the anodic dissolution of iron. Loss of durability in reinforced concrete only occurs if this stable oxide layer is rendered unstable due to ingress of chlorides to the steel /concrete interface or carbonation of the concrete reducing alkaline of the pore solution at the steel/ concrete interface(Zivica, 2003). Diffusion of carbon dioxide from atmosphere or as a permitted additive in form of calcium carbonate in cement can react with the cement hydrates in presence of water to form calcium carbonate and depassivates the oxide layer. On the other hand, the intrusion of chloride ions in reinforced concrete can cause steel corrosion if oxygen and moisture are also available to sustain the reaction. Chloride ions may be introduced into the concrete through intentional inclusion as an accelerating admixture, accidental inclusion as contaminants on aggregates; or penetration by deicing salts, industrial brines; marine spray, fog or mist.The usual corrosion products are oxide, hydroxide, carbonate, sulfide etc.

Although steel can corrode by chemical attack, the most common form of corrosion in an aqueous medium is electrochemical. In most cases electron transfer occurs under electro-chemical action in which there is a transfer operation taking place either between two dissimilar metals or between different areas upon a single material. The zone that releases electrons is called anode while the zone accepting those electrons is termed as cathode as in other electrical circuits.Reaction of anodes and cathodes are broadly referred to as “half-cell reaction”.It works with an anode, where electrochemical oxidation takes place; a cathode, where electrochemical reduction occurs; an electrical conductor, and an aqueous medium.At the anode which is the negative pole, iron is oxidized toferrous ions, shown in equation (1), whereasreduction takes place at the cathode. In an acid medium the reaction taking place at the cathode is the reduction of hydrogen ions to hydrogen. However, concrete is highly basic and usually has an adequate supply of oxygen, so the cathodic reaction occurs according to equation (2).

$$Fe \rightarrow Fe^{2+} + 2e^{-} \quad (1)$$

$$\tfrac{1}{2}O_2 + H_2O + 2e^{-} \rightarrow 2OH^{-} \quad (2)$$

A great deal of research was conducted to investigate corrosion behavior of concrete materials and prevention methods. However, most previous research was focused on undamaged (pristine) concrete and comparatively little work was performed to quantitatively describe how these properties change in the presence of damage or sustained loading, which frequently occurs in field applications. Penetration of chlorides and diffusion of carbon dioxide are increased at the places of cracks, which further increases corrosion. Another consequence of crack formation is the development of galvanic cells with anodic and cathodic areas with corrosion at unprotected (anodic) areas. Pitting corrosion and galvanic macro cell formation generate small losses of steel, but create areas with concentrations of strains(Raupach, 1996). It is, therefore, extremely important to know the mechanism of corrosion process at the initial stage, so that it can be minimized to a greater extent and the heavy losses incurred due to corrosion may be greatly reduced.

The purpose of this study is to assess the effect of corrosion in embedded reinforcement after the subjection of pre-load through accelerated corrosion test. Durable reinforced concrete must be designed to resist carbonation and to exclude chlorides from any source. With the view point that a denser paste matrix would offer a better resistance against corrosion initiation, the corrosion behavior of normal concrete and high performance concrete flexural members is compared.By definition, a high performance concrete (HPC)refers to that type of concrete which is treated in such a manner so that the concrete ensures optimum resistance against various degradation processes in a particular type of climate or environment. The HPC was prepared by including supplementary cementitious materials in this study. The results may provide engineers insight into design, condition evaluation, and service life prediction of reinforced concrete structures.

2. EXPERIMENTAL PROGRAM

2.1 Materials, mix design and sample preparation

Three batches of concrete were constructed, one wasreference concrete beam (Ref) incorporating ordinary Portland cement (OPC) as binder, while the other two were high performance concrete beams, treated by supplementary cementitious materials of slag and fly ash, in batches SL and FA, respectively. The mix proportion is shown in Table 1.Stone chips were used as coarse aggregate with maximum size of 20 mm and coarse sand with a fineness modulus of 2.7 was used as the fine aggregate. Both coarse and fine aggregate were saturated with 1% water before mixing. In Batch SL, 35% OPC was replaced by slag, while 25% OPC was replaced by fly ash in Batch FA. The replacement was on the basis of volume batching. Water-to-binder ratio (w/b) was kept 0.5 in all batches. The beams, having dimensions 100 mm (4 in) wide by 150 mm (6 in) high by 1170 mm long, were reinforced with a single standard No. 5 (16 mm diameter) Grade 60 reinforcing steel bar. The clear cover below the bottom surface of reinforcing steel was 20 mm. To avoid excessive corrosion at the ends of the reinforcing bar, the ending 60 mm range of the bar was coated with epoxy.Specimens were moist cured for 28 days in an 85% relative humidity room at an average temperature of 24^{0}C before being subjected to loading.

Table 1: Mixture Proportion (kg/m^3)

Batch	Cement	Slag	Fly ash	Stone chips	Sand	Water
Ref	470	-	-	1180	470	235
SL	305	150	-	1180	470	230
FA	355	-	85	1180	470	220

2.2 Pre-loading setup

The specimens were subjected to four-point loading test by a hydraulic jack. Destructive flexural test was performed with similar batches of beamsto obtain the average ultimate flexural strength. It was found to be 24.2 kN.Then 80%

of the average ultimate flexural strength was applied on the beams under corrosion investigation for the purpose of pre-loading. The load-deflection diagram is shown in Figure 1. Loading was applied in three successive steps. At the first step, the beams were subjected to 50% of their ultimate load and the load was kept stagnant for 10 minutes. Afterwards, the load was increased to 60% and 80% of their ultimate load in the second and third steps, respectively, and was projected sequentially in a similar manner.

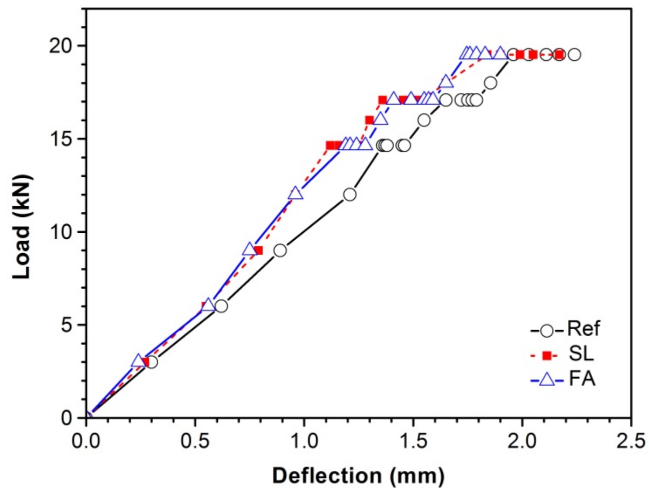

Figure 1: Load-deflection diagram in pre-loading phase

2.3 Corrosion measurement

2.3.1 Galvanic current measurement

The corrosion process of the reinforcing steel in the present study was monitored by the use of galvanic current. A copper plate was placed in a 5% NaCl solution and electrically connected to the reinforcement through a shunt-resistor (2 ohms). The schematic of the corrosion cell is shown in Figure 2. This type of corrosion cell, composed of two dissimilar metals in contact and sharing a common electrolyte (concrete pore solution), is called a galvanic cell(Devalapura, et al., 1994). Among the two dissimilar metals, the metal with the more negative standard potential value serves as anode, while the noble metal with the less negative standard potential value serves as cathode(Berkeley and Pathmanaban, 1990). In this project, mild steel (E = -0.61 Volts) served as anode and copper (E = -0.36 Volts) served as cathode.

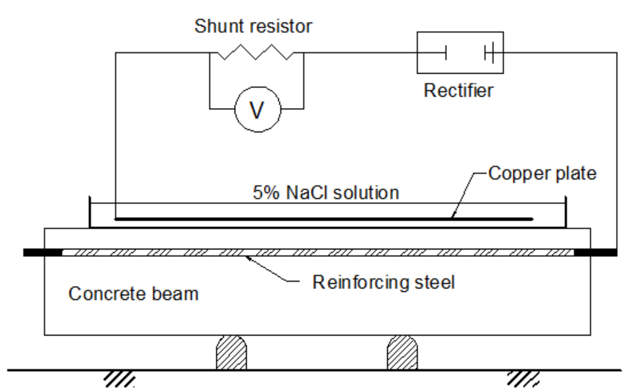

Figure 2: Schematic of galvanic current measurement

The amount of galvanic current, that flows between the galvanic couple – mild steel and copper, indicates the degree of corrosion activity in the cell. The galvanic corrosion current can be calculated from equation (3).

$$I = \frac{V}{R} \tag{3}$$

2.3.2 *Application of external current*

After being subjected to same level of loading (Figure 1), the beams were connected in parallel for the application of an external direct current through an adapter. The complete test setup is shown in Figure 3. The external current was applied in two phases. After 40 days of successive ponding of the concrete surface near the tension zone,a stagnant voltage (9 volt) was applied,in the first phase,to the group of three beams (Ref, SL and FA)for 10 days to initiate the corrosion process. Afterwards, the external direct current was increased at a constant rate to the specimens through an adapter to accelerate the corrosion propagation process. The adapter had one end connected to the reinforcing steel in the specimens while the other endwas connected to the copper plate via a shunt resistor. Then, the voltage was gradually increased (3 volt for each consecutive day) to 30 volts within 8 days in order to obtain a decent degree of corrosion propagation.

$$Weight\ loss = \frac{TC\ x\ EW}{F} \tag{4}$$

Where,
TC = Total electric charge (coulombs)
EW = Equivalent weight that is oxidized (gm)
F = Faraday's constant (96490 coulombs)

Figure 3: Test setup for accelerated corrosion

3. RESULTS AND DISCUSSIONS

3.1 Effect of pre-loading on corrosion initiation

The effect of pre-loading on corrosion initiation and propagation is shown in Figure 4. It compares the corrosion current density and corrosion induced cumulative weight loss (%) between a pristine (undamaged) beam and a pre-loaded beam under the same exposure conditions (5% NaCl ponding). The loading level was 60% of the ultimate load (Figure 1). Measurements were taken twice a day so that corrosion process could be monitored as a function of time. External voltage application was started after 28 days of moist curing and 40 days ofNaCl ponding. A stagnant voltage of 9 volts were kept constant for the first 10 days of the observation and the corrosion current reached to 0.317 $\mu A/cm^2$ in the pre-loaded beam, which was considered to be the threshold for corrosion initiation(Gonzalez, et al., 1980). A rapid change in the corrosion propagation was observed after 11 days when the applied voltage was successively increased. The rapid corrosion propagation might have resulted from larger and higher number of cracks and microcracks that generated in the concrete during loading, which permit chloride penetration for corrosion propagation.

It can be further observed that the specimen without loading had approximately a constant current density, in the range of 0.04-0.26 $\mu A/cm^2$, which indicated that no corrosion of the reinforcement had occurred during the 18 days of accelerated corrosion process.

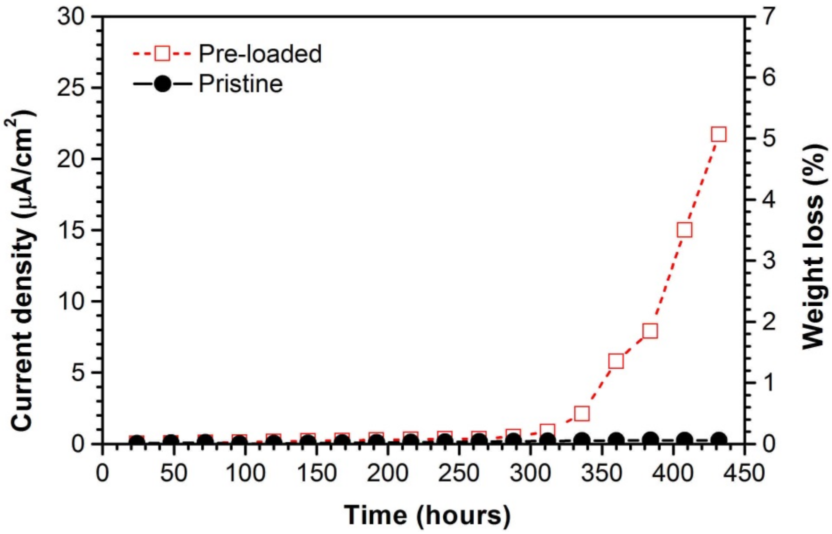

Figure 4: Influence of pre-loading on corrosion process

3.2 Corrosion behaviour of HPC

The accelerated corrosion process between the reference concrete and the high performance concrete beams was compared in Figure 5. It can be seen that the effect of preloading on corrosion propagation was not distinct at the beginning of the corrosion acceleration; but it became more obvious at the later stage. It was possible that some micro-cracks in the concrete that were generated during the pre-loading phase might be closed or became smaller and less connected when the load was removed, thus slowing down further chloride penetration and reducing rate of corrosion propagation (Yoon, et al., 2000). A critical crack width below which the crack does not affect the corrosion process of reinforcing steel was suggested to be approximately between 0.1 and 0.3 mm (Vidal, et al., 2004). Moreover, the relatively smaller cracks might have been sealed by the corrosion products and postponed corrosion propagation.

Among the three specimens, Beam FA where 25% of the OPC was replaced by fly ash had shown lower corrosion current density and cumulative weight loss than the other two specimens. Fly ash has a very rounded particle shape, including some partly broken hollow spheres known as cenospheres (as opposed to the extremely jagged particle shape of cement) and is of lower density (specific gravity usually 1.9 to 2.4 compared to 3.15 for cement).When cement hydrates, it releases free lime. This lime is the softest, weakest and most chemical attack and leaching susceptible of all the constituents of concrete. The fly ash combines chemically with the free lime to form compounds similar to those produced by the rest of the cement. This reaction is quite slow (7 days before it produces much effect), and generated little heat during the setting process, which might have contributed towards a lesser microcracks development during the early days of hydration. Concrete porosity in such concrete could be reduced significantly with increased curing times and, corresponding corrosion resistance is improved.

The concrete beam with slag replacement (SL), showed lesser weight loss in accelerated corrosion than the reference concrete Beam-Ref (Figure 5). Concrete using slag normally has a greater resistance to chemical attack, and is particularly suggested for marine works, however, itgenerally develops strength more slowly than OPCconcrete. Its greater fineness confers resistance to bleeding in the fresh state and lower permeability when hardened.A large amount of ‘free lime’ (CH) is liberated during the hydration of cement. Slag reacts with that hydrated lime, forming a secondary calcium-silicate compound. At the same time, a lot of homogeneous hydration products like ettringite and CH are also formed, which have larger specific surface than that of OPC. In addition, when slag (with high CaO and MgO content) hydrates, it provides OH^{-} ions and alkalis into the pore fluid, which react with SiO_2 and break down the glass phase of slag, which accelerates the process of hydration.Increased corrosion resistance came from the accelerated hydration resulting a densified paste matrix and reduction of permeability, which inhibited penetration of chloride ions.

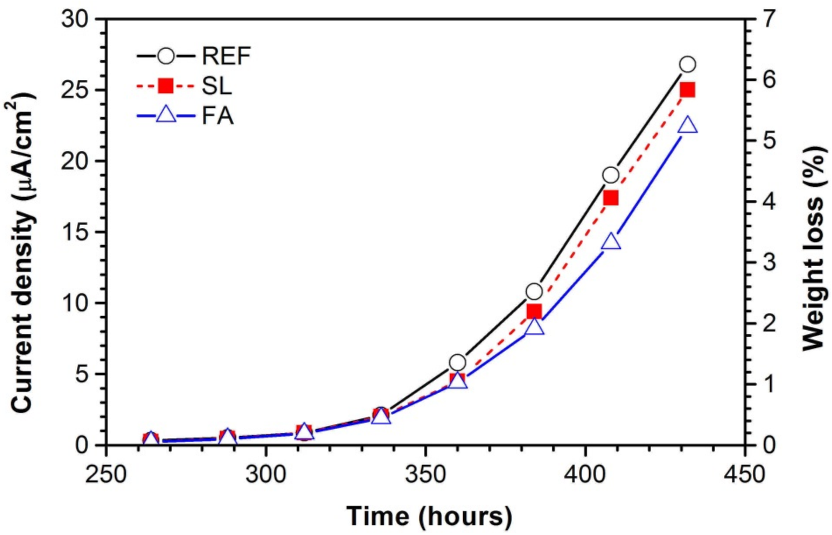

Figure 5: Corrosion process in high performance concrete

4. CONCLUSION

Accelerated corrosion process in high performance concrete beams under preloading condition were investigated in the present study. It involved 28 days of moist curing, 40 days of 5% NaCl ponding and 18 days external current application from corrosion initiation and propagation. Two high performance concrete beams were prepared by replacing a portion of the cement and compared their corrosion resistance performance with a reference concrete beam for the same exposure condition. Test results revealed that corrosion initiation followed liner trend for a constant supply of external current however switched to an exponential trend when the applied current was increased. Concretes having fly ash and slag showed better corrosion resistance in terms of lower current density and lesser cumulative weight loss. Subjection of preloading showed significant influence on corrosion initiation and should be considered for service life modelling.

REFERENCES

Berkeley, K. and Pathmanaban, S. (1990). Cathodic protection of reinforcement steel in concrete. Butterworths and Company Publishers, Limited, Borough Green, Sevenoaks, Kent TN15 8PH England.

Devalapura, R., Kamel, M., and Arumugasaamy, P., (1994) "Nondestructive Techniques for Corrosion Evaluation of Steel in Concrete, Corrosion and Corrosion Protection of Steel in Concrete", Proceedings of International Conference held at the University of Sheffield, Sheffield, UK, Vol. 1, pp. 300-309.

Gonzalez, J.A., Algaba, J., and Andrade, C., (1980) "Corrosion of reinforcing bars in carbonated concrete", British Corrosion Journal, Vol. 15 (3), pp. 135-139.

Raupach, M., (1996) "Chloride-induced macrocell corrosion of steel in concrete—theoretical background and practical consequences", Construction and Building Materials, Vol. 10 (5), pp. 329-338.

Vidal, T., Castel, A., and Francois, R., (2004) "Analyzing crack width to predict corrosion in reinforced concrete", Cement and Concrete Research, Vol. 34 (1), pp. 165-174.

Yoon, S., Wang, K., Weiss, W.J., and Shah, S.P., (2000) "Interaction between loading, corrosion, and serviceability of reinforced concrete", ACI Materials Journal, Vol. 97 (6), pp. 637-644.

Zivica, V., (2003) "Corrosion of reinforcement induced by environment containing chloride and carbon dioxide", Bulletin of Materials Science, Vol. 26 (6), pp. 605-608.

EFFECT OF INDUCED VIBRATION ON FRESH CONCRETE

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ABSTRACT

Concrete is an artificial kind of stone prepared from an indefinite mixture of three basic components such as binding materials, inert materials and water. When the components are mixed together to prepare concrete, binding materials such as cement paste acts as a glue binding surrounding the inert materials during the process of hydration and curing. The freshly mixed concrete becomes hard with considerable strength following the setting procedure of cement. During the setting period, vibration may come on fresh concrete as a result of earthquake, pile driving, heavy traffic or vibratory soil compactors. The objective of this research was to perform a laboratory test to find out the effect of vibration on working strength of concrete between the initial and final setting period. On this purpose, two different sizes concrete cylinders (4"X8" and 6"X12") were casted by mixing cement, sand, stone chips and water with two ratios (1:2:4 and 1:1.5:3). Water-cement ratio was 0.78. The initial and final setting time were recorded as 140min and 270 min respectively. The concrete cylinders were then subjected to two level of vibration for 1 min or 2 min at three different ages (2 hr, 3.5 hr and 5 hr) by the help of shaking table. The peak particle velocities were 50mm/s -100mm/s, 150mm/s and 200mm/s-300mm/s. About 96 cylinders were tested for compression and splitting tensile strength tests at 7 days and 28 days. From the test results it was found that, the compressive strength was increased up to 20% and the tensile strength was decreased up to 13% over the controlled cylinder specimen by the effect of vibration.

Keywords: Fresh Concrete, Setting time, Vibration, Compressive Strength, Tensile Strength

1. INTRODUCTION

Concrete is a stone like material obtained by carefully proportioned mixture of cement, sand and gravel or other aggregate and water to harden in forms of the shape and dimensions of the desired structure (Aziz, 1995). Insufficient vibration of concrete may result defects in concrete, such as honey combing and voids. Many engineers are of the opinion that partially set concrete should not be disturbed in early days. Strong belief has been there that any disturbance to concrete, like re-vibration in the initial hardening stage makes the concrete deteriorate and loose its strength. After some year Engineers have found it necessary to disturb the hardening process.

The effect of nearby dynamite blasts, pile driving, or the simultaneous operation of machinery or vehicles on fresh or early aged concrete is often questioned. Yet there is no documented record of structural failure from these causes.

When ground shaking activities are carried out near freshly cast concrete it is necessary to control shock vibration in order to avoid damage in concrete. Most shock vibration criteria have been based on past experience and subjective judgment that they are sage limits to adopt. Although it is generally believed that the shock vibration resistance of concrete is proportional to strength of concrete it had not been tested.

2. METHODOLOGY

The purpose of this research was to perform a laboratory test program on the effects of induced vibrations on concrete: before, during or after the setting period. For this purpose, a laboratory program was performed. The laboratory program was for casting (4"X8") concrete cylinders about 96 Nos.

At first, the laboratory tests on the course aggregate and fine aggregates such as unit weight, specific gravity, sieve analysis, moisture content of coarse and setting time and consistency of cement has been performed to prepare concrete. For compression and splitting tensile test (4"X8") concrete cylinders of mix proportion 1:2:4 and 1:1.5:3 were casted.

After casting the specimen, the vibration was applied on the fresh concrete between initial and final setting time. There were 3 variables considered in case of vibrating the concrete:

- Age when vibrated
- Peak particle velocity
- And duration of vibration.

The observed initial and final set times were 140 min and 270 min, respectively. The ages at which groups of cylinders were vibrated were 2 h, 3.5 h and 5 h after water–cement contact. Two levels of vibration were conducted for the test: the lower level (L) with particle velocities of approximately 50–100 mm/ s (2–4 in/s) and the higher level (H) with particle velocities of approximately 200–300 mm/ s (8–12 in/s). Durations of 1 and 2 min were used to represent a range of possible times for vibration. Vibration was conducted by using shaking table.

The American Concrete Institute Method ACI 211.1-9114.5 was followed for making and curing concrete test specimens in the laboratory and all specimens were placed in a lime water bath after 24 h and maintained in the bath until testing. The ratios of concrete mixes were 1:2:4 and 1:1.5:3.

All of the specimens with a particular combination of vibration duration and intensity were vibrated simultaneously on a shaking Table. After conducting vibration, each cylinder was given a serial number. Finally, ASTM C39 and IS 1516:1999 were followed to perform 7- and 28-day compression strength and splitting tensile strength tests, respectively.

Figure 1: Concrete in molds

Figure 2: Cylinders in shaking table

3. ILLUSTRATIONS

Compressive strength (7days)

Peak particle velocity :H=200 mm/s~300 mm/s

L=50 mm/s~100 mm/s

Table 1: Results of compression test at 7 days

Concrete ratio	Age when vibrated	Peak particle velocity	Duration of time	Age when tested	Compressive force	compressive strength	Change over control average
1:2:4	(hour)	(mm/s)	(min)	(days)	(Ib)	(psi)	%
1:2:4	Control	-	-	7	24000	1910	-
1:2:4	2	H	1	7	25000	1988	4
1:2:4	2	H	2	7	29000	2307	20
1:2:4	2	L	1	7	26000	2068	8
1:2:4	2	L	2	7	27000	2148	12
1:2:4	3.5	H	1	7	31000	2466	29
1:2:4	3.5	H	2	7	29000	2307	20
1:2:4	3.5	L	1	7	30000	2386	24
1:2:4	3.5	L	2	7	31000	2466	29
1:2:4	5	H	1	7	23000	1828	-4
1:2:4	5	H	2	7	24000	1910	-
1:2:4	5	L	1	7	30000	2387	24
1:2:4	5	L	2	7	25000	1989	*4*
1:1.5:3	Control	-	-	7	36000	2864	-
1:1.5:3	2	H	1	7	37000	2943	*3*
1:1.5:3	2	H	2	7	39000	3103	*8*
1:1.5:3	2	L	1	7	33000	2625	*-8*
1:1.5:3	2	L	2	7	36000	2864	-
1:1.5:3	3.5	H	1	7	40000	3182	*11*
1:1.5:3	3.5	H	2	7	35000	2785	*-3*

1:1.5:3	3.5	L	1	7	41000	3262	*14*
1:1.5:3	3.5	L	2	7	37000	2943	*3*
1:1.5:3	5	H	1	7	36000	2864	-
1:1.5:3	5	H	2	7	37000	2943	*3*
1:1.5:3	5	L	1	7	39000	3103	*8*
1:1.5:3	5	L	2	7	40000	3182	*11*

Graphical representation

Vibration strengthens the concrete in compression, increasing the compressive strength by as much as 15% over the control cylinders. In figure, the changes in compressive strength compared to controlled cylinder have shown. Results were measured with respect to three variables. In Figure 1 the marked line indicates the control cylinder compressive strength, which was not vibrated.

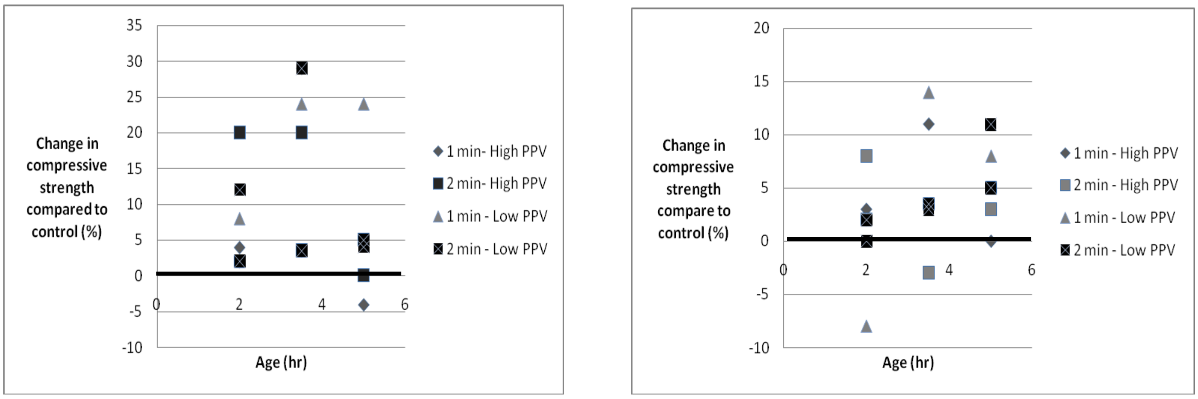


Figure 3: Impact on compressive strength at 7 days for four combination of PPV and duration of induced vibration. Percentages based on average control cylinders, which were not vibrated. (ratio 1:2:4 left; ratio 1:1.5:3 right)

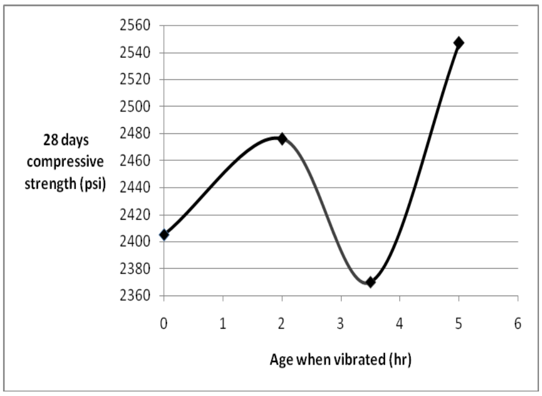
Compressive strength (28 days)

Table 2: Results of compression test at 28 days

Concrete ratio	Age when vibrated (hr)	Duration of vibration (min)	Peak particle velocity (mm/s)	Age when tested (days)	Compressive force (Ib)	Compressive strength (psi)	Change over control average(%)
1:2:4	Control	-	-	28	68000	2405	-
1:2:4	2	1	150	28	70000	2476	3
1:2:4	3.5	1	150	28	67000	2370	-2
1:2:4	5	1	150	28	72000	2547	6
1:1.5:3	Control	-	-	28	105000	3714	-
1:1.5:3	2	1	150	28	107000	3785	2
1:1.5:3	3.5	1	150	28	109000	3856	4
1:1.5:3	5	1	150	28	110000	3891	5

Graphical representation

The same trend can be seen much more dramatically in the 28-day compression test results, where all of the groups of cylinders that were vibrated had stronger average strengths than the control group as seen in Figure 4. In fact only 1 of the cylinders had a weaker compressive strength than the control average. It is clear that, the compressive strength was increased in average because of induced vibration



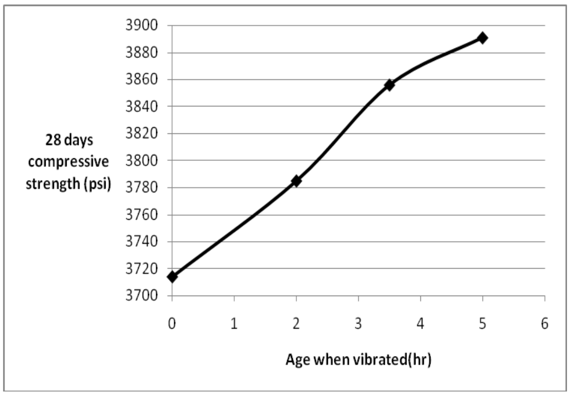


Figure 4: 28 days compressive strength with time comparison (ratio 1:2:4 left and ratio 1:1.5:3 right)

Tensile strength (7days)

Peak particle velocity :H=200 mm/s~300 mm/s
L=50 mm/s~100 mm/s

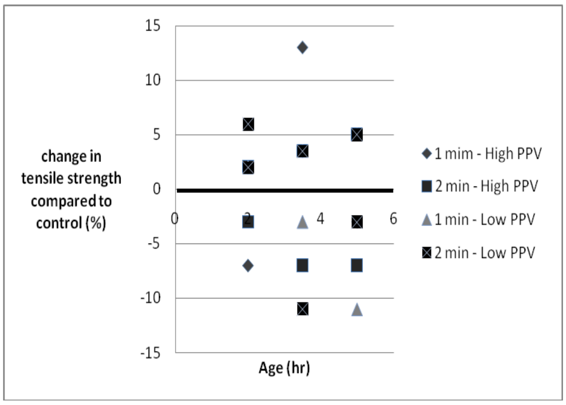
Table 3: Results of tensile strength in 7 days

Concrete ratio	Age when vibrate	Peak particle velocity	Duration of vibration	Age when tested	Force	Tensile strength	Change over control average
1:2:4	(Hour)	(mm/s)	(min)	(days)	(Ib)	(psi)	%
1:2:4	Control	-	-	7	11333	225	-
1:2:4	2	H	1	7	10500	209	-7
1:2:4	2	H	2	7	11000	219	-3
1:2:4	2	L	1	7	11500	229	2
1:2:4	2	L	2	7	12000	239	6
1:2:4	3.5	H	1	7	13000	259	13
1:2:4	3.5	H	2	7	10500	209	-7
1:2:4	3.5	L	1	7	11000	219	-3
1:2:4	3.5	L	2	7	10000	199	-11
1:2:4	5	H	1	7	11000	219	-3
1:2:4	5	H	2	7	10500	209	-7

1:2:4	5	L	1	7	10000	199	-11
1:2:4	5	L	2	7	11000	219	-3
1:1.5:3	Control	-	-	7	17000	338	-
1:1.5:3	2	H	1	7	16500	328	-3
1:1.5:3	2	H	2	7	16000	318	-6
1:1.5:3	2	L	1	7	17500	348	3
1:1.5:3	2	L	2	7	16000	318	-6
1:1.5:3	3.5	H	1	7	15500	309	-8
1:1.5:3	3.5	H	2	7	15000	299	-11
1:1.5:3	3.5	L	1	7	16000	318	-6
1:1.5:3	3.5	L	2	7	15000	299	-11
1:1.5:3	5	H	1	7	18000	358	6
1:1.5:3	5	H	2	7	17000	338	-
1:1.5:3	5	L	1	7	16500	328	-3
1:1.5:3	5	L	2	7	17500	348	3

Graphical representation

In figure 5, it has shown that vibration weakens the concrete tensile strength by up to 13%. The marked line indicates the control cylinder tensile strength, which was not vibrated. So, the tensile strength decreases in average with vibration.



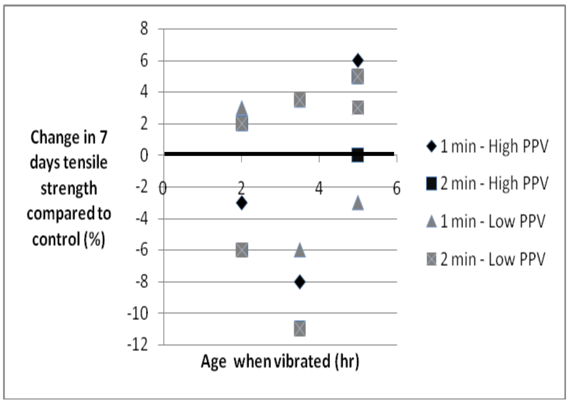


Figure 5 : Impact on tensile strength at 7 days for four combination of PPV and duration of induced vibration. Percentages based on average control cylinders, which were not vibrated (ratio 1:2:4 left and ratio 1:1.5:3 right)

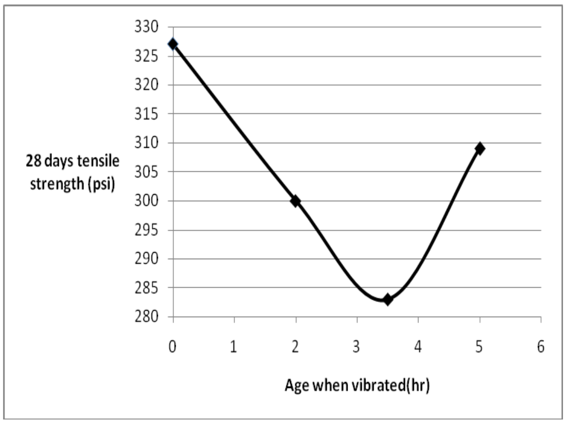
Tensile strength at 28 days

Table 4.6 : Results of tensile strength tests at 28 days

Proportion	Age when vibrated (hr)	Duration of vibration (min)	Peak particle velocity (mm/s)	Age when tested (days)	Tensile force (Ib)	Tensile strength (psi)	Change over control average(%)
1:2:4	Control	-	-	28	37000	327	-
1:2:4	2	1	150	28	34000	300	-8
1:2:4	3.5	1	150	28	32000	283	-13
1:2:4	5	1	150	28	35000	309	-6
1:1.5:3	Control	-	-	28	51000	451	-
1:1.5:3	2	1	150	28	52000	460	2
1:1.5:3	3.5	1	150	28	48000	427	-6
1:1.5:3	5	1	150	28	47000	416	-8

Graphical representation

The same trend can be seen much more dramatically in the 28-day temsile test results, where all of the groups of cylinders that were vibrated had weaker average strengths than the control group as seen in Figure 6.



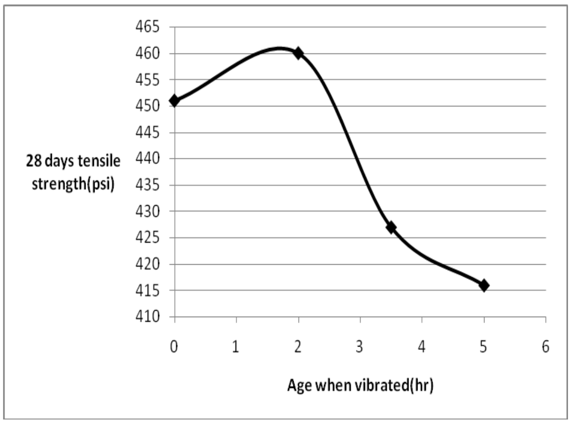


Figure 6 : 28 days time comparison (ratio 1:2:4 left and ratio 1:1.5:3 right)

4. CONCLUSIONS

The compressive strength of the cylinders has increased, but tensile strength has decreased. The vibrations could reduce the water content of the concrete. The final setting period was 4.5 hour. Therefore any cylinders that were vibrated before 4.5 hours could have experienced the effect of lower water content. The vibrations could have strengthened the concrete is if the vibrations closed some of the air voids in the concrete. It has been suggested (Buenfeld 1999) that reduction of air voids in concrete increases the compressive strength. This effect may also explain some of the strength loss in tension strength. The results of this study have shown that vibration under a limit has a positive impact on the ultimate compressive strength of concrete. But it has a slight negative effect on the tensile capacity. The results do not show a strong reason to create any alarm. It could not be decided that it is safe to operate vibratory soil compactors adjacent to setting foundations. Further more studies can determine the limitations of vibrating soil near setting concrete.

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REFERENCES

Michael R.; Adam S.; and James H. (2007),” Effects of Induced Vibrations on Early Age Concrete”.

Krishna R., Kumar R. and Bhaskar B. (2008),” Effect of Re-Vibration on Compressive Strength of Concrete”.

CONCRETE BASICS A Guide to Concrete Practice, (2006), Cement Concrete & Aggregates Australia.

Aziz, M.A. 1995. A Text Book of Engineering Materials. 167, Dhaka New Market, Dhak-1205: Hafiz Book Centre.

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STRUCTURAL PERFORMANCE INVESTIGATION BASED ON SIMPLE ASSESSMENT PROCEDURE AT CUET CAMPUS

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ABSTRACT

Bangladesh is situated in the seismically moderate risky zone due to its geographical location. Presences of exiting fault lines surroundings the country are capable to produce damageable earthquake. Frequently occuring light earthquakes in last couple of years make us aware about its future risk. In proposed Bangladesh National Building Code (BNBC) Second Edition seismic map, Chittagong city is quite subtancial to earthquake with a seismic zoning coefficient of 0.28. One of the leading technical universities of the country, Chittagong University of Engineering and Technology (CUET) campus is located about 27 km far from the heart of the Chittagong city center. This study was undertaken to prepare a structural database and to ivestigate the seismic safety of the structures by applying a two-level based simple assessment technique. Rapid screening procedure (RSP) was peformed in the first tire of assessment. Then Structural Inegrity Check was performed for the academic and administrative buildings in the second tire. Structural information database was prepared and presented in Geographic Information System (GIS). Most of these buildings performed well in the both level of assessments.

Keywords: Assessment, CUET, Rapid Screening Procedure, Performance, Vulnerability.

1. INTRODUCTION

Bangladesh is situated in a moderate seismic region in the world seismic map according to Global Seismic Hazard Assessment Program (GSHAP, 1992). An earthquake of even medium magnitude can produce massive destruction in major urban areas of the country, especially Dhaka, Sylhet and Chittagong. Recent earthquakes in India, Pakistan and Myanmar make us aware about the future risk of this region. Great Inidan Earthquake in 1897 with a Richter Local Magnitude 8.7 is the previous large event triggered at Shillong near the Sylhet-Meghalaya border area. No earthquake occurred in these faults for many years, which means huge strength has gathered underground that could cause serious earthquakes in Bangladesh and its neighboring areas any time. There exist few faults in this region that can cause strong earthquake in the country. One of them is the Dauki fault at the bordering area of Sylhet–and the other one is Sitakunda-Taknaf fault at Chittagong coastal area. However, a recent study conducted in Comprehensive Disaster Management Program (CDMP) proposed five earthquake scenarios where each scenario was set as a maximum possible earthquake (Mw) occurring within a fault zone. These five major fault zones are named as Madhupur fault, Dauki Fault, Plate Boundary Fault -1, Plate Boundary Fault -2 and Plate Boundary Fault -3 (as shown in Figure 1). A special earthquake scenario where a magnitude-6 earthquake is occurring beneath Chittagong city was recommended during the study (CDMP, 2009). The scenario of falt parameters for the Chottagong city was shown in Table 1.

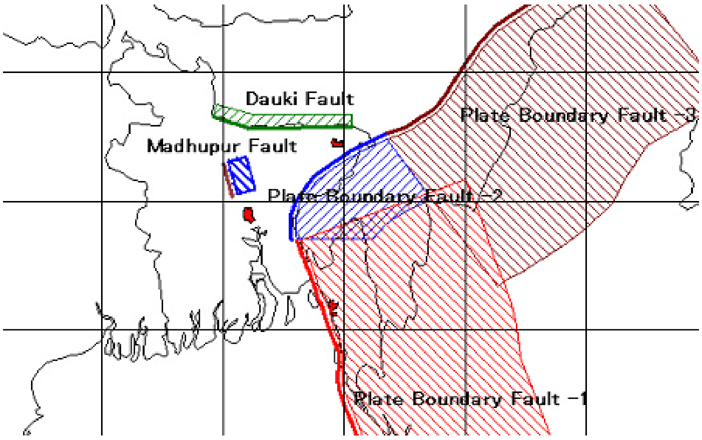


Figure 1: Earthquake Fault Model in Bangladesh, (CDMP, 2009).

Table 1: Earthquake Scenario Parameters for Chottagong city area.

Case	Coordinate of Epicenter		M_w	Depth to top of fault (km)	Dip Angle	Fault type	Description
	Latitude	Longitude					
1	21.1	92.1	8.5	17.5	30°	Reverse	Plate Boundary Fault -1
2	23.8	91.1	8.0	3	20°	Reverse	Plate Boundary Fault -2
3	22.4	91.8	6.0	25	90°	Reverse	M_w 6.0 beneath the city

Chittagong University of Engineering & Technology (CUET) is one of the leading technical institutions in the country which is located by the side of the Chittagong-Kaptai road and about 27 kilometers off from the Chittagong city center. This region falls into zone number 2 in the Bangladesh National Building Code (BNBC 1993) with a seismic coefficient of 0.15 g. This map was based on Peak Ground Acceleration (PGA) considering a return period of 200 years. In proposed Bangladesh National Building Code (BNBC) Second Edition seismic map, the concept of Maximum Credible Earthquake (MCE) has been inreoduced where MCE is correspond to a return period of 2475 means 2% probability of exceedance in 50 years. In this new seismic map, CUET falls into zone number 3 with a coefficient of 0.28 g.

It has been felt necessary to prepare a structural database of existing buildings at CUET campus. This study was undertaken by the Institute of Earthquake Engineering Research (IEER) to assess the seismic safety of existing structures by applying two-level based seismic vulnerability assessment techniques. First tire includes simple walkdown assessment by visualizing the structural vulnerability parameters. In the second tire, buildings were assessed by checking their structural integrity.

2. METHODOLOGY

The medthod was originally generated by Ozcebe et al. In 2006. It is very important to identify seismically vulnerable building before taking any strengthening measure. To survey all buildings in detail level is neither feasible nor possible, Rapid Screening Procedure is widely accepted before considering any structural detail level of investigation. To evaluate the seismic condition of the existing buildings, two level risk assessment procedures applied based on the structures level of importance in terms of building use. Mainly two major types of structures are presence at CUET. One type is Reinforce Concrete (RC) frame structures with masonry infill and another is Unreinforced Masonry Buildings (URM) with flexible or rigid diaphragm. For a RC building, Turkish simple screening procedure (Ozcebe et al., 2006) was followed. This procedure contains three level of assessment whre only RC building can be analyzed. Detail structural assessment wasn't performed due to insufficient informations (eg., destructive, non-destructive tests results). Rapid Visual Screening (FEMA 154) was applied for masonry structures.

2.1 Turkish Method

2.1.1 Tire 1 Assessment

A street survey procedure based on simple structural and geotechnical parameters that can be observed easily from the sidewalk. The time required for an observer for collecting the data of one building from the sidewalk is expected about 20 minutes. The parameters that are selected for representing building vulnerability in this study are the following:

1) The number of stories above ground (1 to 7)
2) Presence of a soft story (Yes or No)
3) Presence of heavy overhangs, such as balconies with concrete parapets (Yes or No)
4) Apparent building quality (Good, Moderate or Poor)
5) Presence of short columns (Yes or No)
6) Pounding between adjacent buildings (Yes or No)
7) Local soil conditions (Stiff or Soft)
8) Topographic effects (Yes or No)

The intensity of ground motion at a particular site predominantly depends on the distance the causative fault and local soil conditions. As there exists a strong correlation between Peak Ground Velocity and the shear wave velocities of local soils, in this study the PGV is selected as to represent the ground motion intensity. The intensity zones are expressed accordingly, in terms of the associated PGV ranges.

Zone I : $60<PGV<80$ cm/s^2
Zone II : $40<PGV<60$ cm/s^2
Zone III : $20<PGV<40$ cm/s^2

The selected buildings were mainly low-rise buildings with one to five stories above ground. According to proposed seismic map of BNBC code, for CUET campus the peak ground acceleration is around 0.28 g, considering site effects it can be taken as more than 0.30 g. Corresponding PGV can be taken as between 60 cm/s to 70 cm/s (Wu et al., 2003). Hence for calculating performance score Zone I (60<PGV<80) is considered.

Table 2: Base Scores and Vulnerability Scores for Concrete Buildings

Number of Stories	Base Scores(BS)			Vulnerability Scores(VS)					
	Zone I	Zone II	Zone III	Soft Story	Heavy Overhang	ApparentQuality	Short Column	Pounding Effect	Topo. Effects
1 or 2	100	130	150	0	-5	-5	-5	0	0
3	90	120	140	-15	-10	-10	-5	-2	0
4	75	100	120	-20	-10	-10	-5	-3	-2
5	65	85	100	-25	-15	-15	-5	-3	-2
6 or 7	60	80	90	-30	-15	-15	-5	-3	-2

The vulnerability parameters of a building are obtained from Walkdown surveys and its location is determined, the seismic performances score (PS) can be calculated by using Eqn. 2.1

$$PS = (BS) - \Sigma\,(VSM) \times (VS) \quad (2.1)$$

Where BS is the Base Score defined in Table 2, Σ (VSM) is the Summation of Vulnerability Score Multiple and VS is Vulnerability Scores.

2.1.2 Tire 2 Assessment

The following parameters were chosen as the basic estimation parameters in the preliminary assessment level.

1) No of stories (n): this is the total number of individual floor systems above the ground level defined by ‘n’.
2) Minimum normalized lateral stiffness index (mnlstfi): this index represents the lateral rigidity of the ground story, which is usually the most critical story. It is calculated by considering the columns and the structural walls at the ground story. The mnlstfi parameter shall be computed based on the following relationship:

$$mnlstfi = min\,(I_x, I_y) \quad (2.2)$$

where;

$$I_{nx} = \frac{\sum (I_{col})_x + \sum (I_{sw})_x}{\sum A_f} \times 1000 \quad , \quad I_{ny} = \frac{\sum (I_{col})_y + \sum (I_{sw})_y}{\sum A_f} \times 1000 \qquad (2.3)$$

Where, $\sum(I_{col})_x$ and $\sum (I_{col})_y$ are the summation of the moment of inertias of all columns about their censorial x and y axes, respectively. $\sum (I_{sw})_x$ And $\sum (I_{sw})_y$ are the summation of the moment of inertias of all structural walls about their censorial x and y axes, respectively. I_{nx} and I_{ny} are the total normalized moment of inertia of all members about x and y axes, respectively. $\sum A_f$ is the total floor area above ground level.

3) Minimum normalized lateral strength index (mnlsi)

It indicates the base shear capacity of the critical story. In the calculation of this index, unreinforced masonry filler walls are assumed to carry 10 percent of the shear force that can be carried by a structural wall having the same cross-sectional area (Sozen, 1997). As in mnlstfi calculation, the vertical reinforced members with a cross-sectional aspect ratio of 7 or more are classified as structural walls. The mnlsi parameter shall be calculated by using the following equation:

$$mnlsi = min\ (A_{nx}, A_{ny}) \qquad (2.4)$$

where:

$$A_{nx} = \frac{\sum (A_{col})_x + \sum (A_{sw})_x + 0.1\sum (A_{mw})_x}{\sum A_f} \times 1000 \quad , \quad A_{ny} = \frac{\sum (A_{col})_y + \sum (A_{sw})_y + 0.1\sum (A_{mw})_y}{\sum A_f} \times 1000 \qquad (2.5)$$

For each column with a cross-sectional area denoted by A_{col}:

$$(A_{col})_x = K_x .\ A_{col} (A_{col})_y = K_y .\ A_{col} \qquad (2.6)$$

where; k_x=1/2 for square and circular columns; k_x=2/3 for rectangular columns with b_x>b_y; k_x=1/3 for rectangular columns with b_x<b_y; and k_y=1-k_x

For each shear wall with cross-sectional area denoted by A_{sw}:

$$(A_{sw})_x = K_x.A_{sw} (A_{sw})_x = K_x.A_{sw} \qquad (2.7)$$

where; k_x=1 for structural walls in the direction of x-axis; k_x=0 for structural walls in the direction of y-axis; and k_y =1-k_x

For each unreinforced masonry filler wall with no window or door opening and having across-sectional area denoted by A_{mw}:

$$(A_{mw})_x = K_x .A_{mw} (A_{mw})_x = K_x .A_{mw} \qquad (2.8)$$

where; k_x=1.0 for masonry walls in the direction of x-axis; k_x=0 for masonry walls in the direction of y-axis; and k_y=1-k_x

4) Normalized redundancy score (nrs)

Redundancy is the indication of the degree of the continuity of multiple frame lines which distribute lateral forces throughout the structural system. The normalized redundancy ratio (nrr) of a frame structure is calculated by using the following expression:

$$nrr = \frac{A_{tr}(nf_x - 1)(nf_y - 1)}{A_{gf}} \qquad (2.9)$$

where; A_{tr} = the tributary area for a typical column. (A_{tr} shall be taken as 25 m^2 if nf_x and nf_y are both greater than and equal to 3. In all other cases, A_{tr} shall be taken as 12.5 m^2); nf_x = the number of continuous frame lines in the critical story (usually the ground story) in x directions; nf_y = the number of continuous frame lines in the critical story (usually the ground story) in y directions; A_{gf} = the area of the ground story, i.e. the footprint area of the building. Depending on the value of nrr computed from Eqn. 2.9, the following discrete values are assigned to the normalized redundancy score (nrs):

nrs = 1 for 0 <nrr ≤ 0.5
nrs = 2 for 0.5 <nrr ≤ 1.0
nrs = 3 for 1.0 >nrr

5) Soft story index (ssi)

On the ground story, there are usually fewer partition walls than in the upper stories. This situation is one of the main reasons for the soft story formations. Since the effects of masonry walls are included in the calculation of mnlsi, soft story index is defined as the ratio of the height of first story (i.e. the ground story), H_1, to the height of the second story, H_2.

$$ssi = \frac{H_1}{H_2} \tag{2.10}$$

6) Overhang ratio (or)

In a typical floor plan, the area beyond the outermost frame lines on all sides is defined as the overhang area. The summation of the overhang area of each story, $A_{overhang}$, divided by the area of the ground story, A_{gf}, is defined as the overhang ratio.

$$or = A_{Overhang}/A_{gf} \tag{2.11}$$

7) Performance Classification:

The Damage Index (DI) or the damage score corresponding to the life safety performance classification (LSPC) shall be computed from the discriminate function given in Eqn. 2.12.

$$DI_{LS} = 0.620n - 0.246mnlstfi - 0.182mnlsi - 0.699nrs + 3.269ssi + 2.728or - 4.905 \tag{2.12}$$

In the case of immediate occupancy performance classification (IOPC), the damage index can be computed based on the following Eqn. 2.13.

$$DI_{IO} = 0.808n - 0.334mnlstfi - 0.107mnlsi - 0.687nrs + 0.508ssi + 3.884or - 2.868 \tag{2.13}$$

In the proposed classification methodology, buildings are evaluated according to both performance levels. The steps to be followed are listed below. The Cutoff Value (CV) for each performance classification can be calculated using Eqn. 2.14. The LS_{CVR} and IO_{CVR} values shall be obtained from Table 3, based on the number of stories above the ground level. The Cutoff Modifier Coefficient (CMC) values are adjustment factors, which introduce the spatial variation of the ground motion in the evaluation process. These values shall be taken from Table 4, based on the building location relative to the fault and the soil type at the site.

$$CV_{LS} = LS_{CVR} + |LS_{CVR}| \times (CMC-1),$$
$$CV_{IO} = IO_{CVR} + |IO_{CVR}| \times (CMC-1) \tag{2.14}$$

Table 3: Variation of LS_{CVR} and IO_{CVR} values with number of stories

n	LS_{CVR}	IO_{CVR}
3 or Less	0.383	-0.425
4	0.430	-0.609
5	0.495	-0.001
6	1.265	0.889
7	1.791	1.551

Table 4: Variation of CMC values with soil type and distance to fault

Soil Type	Shear Wave Velocity(m/s)	Distance to Fault (km)				
		0-4	5-8	9-15	16-25	>26
B	>760	0.778	0.824	0.928	1.128	1.538
C	360-760	0.864	1.000	1.240	1.642	2.414
D	180-360	0.970	1.180	1.530	2.099	3.177
E	<180	1.082	1.360	1.810	2.534	3.900

(3) By comparing the CV values with associated DI value calculate performance grouping of the building for LSPC and IOPC as follows:
If $DI_{LS}>CV_{LS}$ take $PG_{LS}=1$
If $DI_{LS}<CV_{LS}$ take $PG_{LS}=0$
If $DI_{IO}>CV_{LS}$ take $PG_{IO}=1$
If $DI_{IO}<CV_{LS}$ take $PG_{IO}=0$

To decide the probable expected performance level of the building the damage scores obtained from equation 2.12 & 2.13 should be compared with the story dependent cut off values obtained from equation 2.14. In each case, the building under evaluation is assigned an indicator variable of '0' or '1'. The indicator variable '0' corresponds to 'none, light or moderate damage' in the case of LSPC and 'none or light damage' in the case of IOPC. Similarly, the indicator variable '1' corresponds to 'severe damage or collapse' in the case of LSPC and 'moderate or severe damage or collapse' in the case of IOPC. In the final stage, the building is rated in the 'low risk group' if both indicator values are zero or in the 'high risk group' when both indicator values are equal to unity. In all other cases buildings are classified as the cases 'requiring further study. Further investigations have indicated that these buildings generally lie in the moderate risk group.

2.2 Rapid Visual Screening

Rapid visual screening (RVS) of buildings for potential seismic hazards, originated in1988 with the publication of the Federal Emergency Management Agency (FEMA) 154 Report, Rapid Visual Screening of Buildings for Potential Seismic Hazards a handbook (FEMA, 1988). RVS provides a procedure to identify record and rank buildings that are potentially seismically hazardous. This screening methodology is encapsulated in a one-page form, which combines a description of a building, its layout and occupancy, and a rapid structural evaluation related to its seismic hazard. The RVS has been developed for a broad audience, including building officials and inspectors, and government agency and private-sector building owners, to identify, inventory, and rank buildings that are potentially seismically hazardous. Field screening of individual buildings is consists of verifying and updating building identification information, walking around the building and sketching a plan and elevation view on the survey forms, determining occupancy class, number of occupants, collecting information of soil type, identifying potential nonstructural falling hazards, lateral-load-resisting system seismic performance attribute score modifiers (e.g., number of stories, design date) and determining the final Score by adjusting the basic structural hazard score with the score modifiers. The final score is the deciding factor that further evaluation is required or not. Photograph of the building is required to justify the buildings properly. FEMA 154 has three seismic zones where Chittagong region is fall into moderate seismic zones for the short period structures with spectral acceleration less than 1.0 sec. As the most of the buildings are less than 5 stories, study area fall moderate seismic zone for RVS application. Table 5 represents the seismic regions classification based on acceleration response. Soil type was considered as D in the FEMA 154 handbook considering experts opinion. Figure 2 shows an example of score modifiers for performance score calculation. Fundamentally, the final S score is an estimate of the probability (or chance) that the building will collapse if ground motions occur that equal or exceed the maximum considered earthquake ground motions These estimates of the score are based on limited observed and analytical data, and the probability of collapse is therefore approximate. A final score of S = 2 implies there is a chance of 1 in 10^2, or 1 in 100, that the building will collapse if such ground motions occur. According to RVS procedure the vulnerability parameters are: a) apparent quality, b) vertical irregularity, c) pre code, d) post benchmark year/age, e) local soil type, f) structural system and g) plan irregularity.

BASIC SCORE, MODIFIERS, AND FINAL SCORE, *S*

BUILDING TYPE	W1	W2	S1 (MRF)	S2 (BR)	S3 (LM)	S4 (RC SW)	S5 (URM INF)	C1 (MRF)	C2 (SW)	C3 (URM INF)	PC1 (TU)	PC2	RM1 (FD)	RM2 (RD)	URM
Basic Score	**5.2**	**4.8**	**3.6**	**3.6**	**3.8**	**3.6**	**3.6**	**3.0**	**3.6**	**3.2**	**3.2**	**3.2**	**3.6**	**3.4**	**3.4**
Mid Rise (4 to 7 stories)	N/A	N/A	+0.4	+0.4	N/A	+0.4	+0.4	+0.2	+0.4	+0.2	N/A	+0.4	+0.4	+0.4	-0.4
High Rise (>7 stories)	N/A	N/A	+1.4	+1.4	N/A	+1.4	+0.8	+0.5	+0.8	+0.4	N/A	+0.6	N/A	+0.6	N/A
Vertical Irregularity	-3.5	-3.0	-2.0	-2.0	N/A	-2.0	-2.0	-2.0	-2.0	-2.0	N/A	-1.5	-2.0	-1.5	-1.5
Plan Irregularity	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5
Pre-Code	0.0	-0.2	-0.4	-0.4	-0.4	-0.4	-0.2	-1.0	-0.4	-1.0	-0.2	-0.4	-0.4	-0.4	-0.4
Post-Benchmark	+1.6	+1.6	+1.4	+1.4	N/A	+1.2	N/A	+1.2	+1.6	N/A	+1.8	N/A	2.0	+1.8	N/A
Soil Type C	-0.2	-0.8	-0.6	-0.8	-0.6	-0.8	-0.8	-0.6	-0.8	-0.6	-0.6	-0.6	-0.8	-0.6	-0.4
Soil Type D	-0.6	-1.2	-1.0	-1.2	-1.0	-1.2	-1.2	-1.0	-1.2	-1.0	-1.0	-1.2	-1.2	-1.2	-0.8
Soil Type E	-1.2	-1.8	-1.6	-1.6	-1.6	-1.6	-1.6	-1.6	-1.6	-1.6	-1.6	-1.6	-1.6	-1.6	-1.6
FINAL SCORE *S*															

COMMENTS	Detailed Evaluation Required YES NO

* = Estimated, subjective, or unreliable data
DNK = Do Not Know
BR = Braced frame
FD = Flexible diaphragm
LM = Light metal
MRF = Moment-resisting frame
RC = Reinforced concrete
RD = Rigid diaphragm
SW = Shear wall
TU = Tilt up
URM INF = Unreinforced masonry infill

Figure 2: Basic score modifiers for final score calculation (moderate seismicity zone).

Table 5: Regions of Seismicity with corresponding spectral acceleration response.

Region of Seismicity	Spectral Acceleration Response, SA in horizontal direction (short-period, or 0.2 sec)	Spectral Acceleration Response, SA in horizontal direction (long-period or 1.0 sec)
Low	Less than 0.167 g	Less than 0.067g
Moderate	Greater than or equal to 0.167 g but less than 0.500g	Greater than or equal to 0.067 g but less than 0.200g
High	Greater than or equal to 0.500 g	Greater than or equal to 0.200 g

Notes: g = acceleration of gravity

3. ANALYSIS AND RESULTS

Total 61 existing buildings considered for this study. However, other existing buildings under construction weren't taken into consideration. All of the buildings are less than 6 stories. Table 6 represents the number of buildings exits according to their story numbers. The table reflects that 91.8 percent buildings are less than 4 stories. Among the surveyed buildings, 56 percent buildings are RC structures, 38 percent buildings are masonry with rigid diaphragm and rests of buildings are masonry with flexible diaphragm (see in Figure 3a). Table 8 shows the relationship exiting in between building structural types and number of stories.

Table 6: Buildings according to number of stories

Number	≤ 2	3	4	5	6	7	7+	Total
Number	36	20	4	1	0	0	0	61

Table 7: Different structural types presence in CUET

RC frame	Masonry with rigid diaphragm	Masonry with flexible diaphragm	Total
34	23	4	61

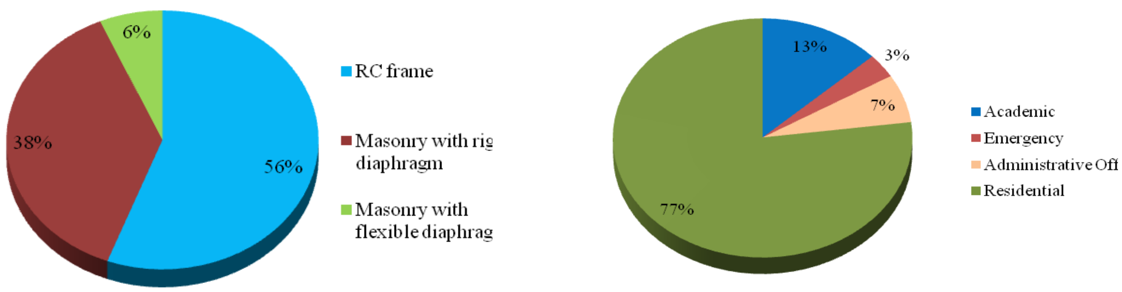


Figure 3: a) Presence of different building structural types and b) occupancy classes.

Table 8: Relationship between building structural types and number of stories.

Number of Stories	RC	Masonry with flexible diaphragm	Masonry with rigid diaphragm	Total
1	6	18	4	28
2	8	0	0	8
3	16	4	0	20

4	3	1	0	4
5	1	0	0	1
Total	34	23	4	61

Total buildings are classified into four categories based on their purpose of uses. Figure 3b rrepresents exiting building use categories in percentage. Majority numbers of the buildings are using for the residential purposes. Only 13 percent buildings use for the academic purposes, 7 percent buildings are administrative and 3 percent buildings are emergency center. Table 9 shows the relationship between building occupancy class and number of stories.

Table 9: Building occupancy class varies with building number stories.

Number of Stories	Academic	Emergency	Administrative Office	Residential	Total
1	4	2	1	21	28
2	1	0	1	6	8
3	3	0	2	15	20
4	0	0	0	4	4
5	0	0	0	1	1
Total	8	2	4	47	61

3.1 Level 1 Assessment

First stage assessment was basically walkdown procedure consists of Turkish tire 1 and RVS. Turkish level 1 survey method used for 34 RC structures. Figure 4a represents the existing physical visible condition of the buildings in percentile form. Table 10 shows the relationship of the buildings apparent quality varies with building number of stories.

Table 10: Building Physical Condition

Number of Stories	Good	Moderate	Poor	Total
1	14	12	2	28
2	1	7	0	8
3	0	19	1	20
4	0	4	0	4
5	1	0	0	1
Total	16	42	3	61

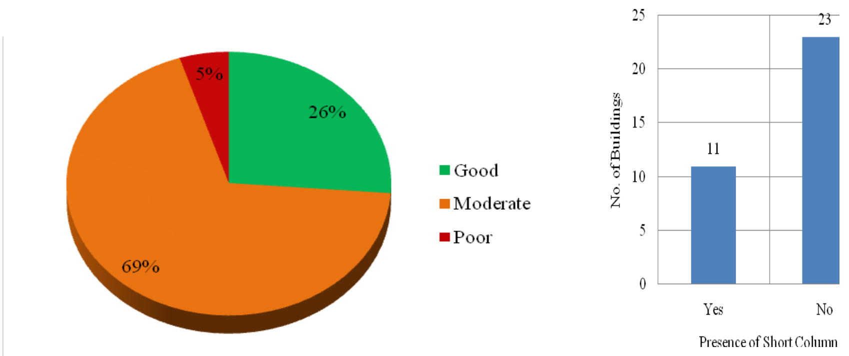


Figure 4: a) Building's physical visible condition and b) presence of short column.

CUET is geographically situated in the hilly regions so that topographic effects considered for the study. There has no soft story presence in the existing buildings at CUET. Figure 4b represents number of buildings presence having short column effect. Table 11 shows short column presence with respect to different number of stories. Heavy overhang and pounding effect presence for a single building over the campus region. The 5 storied hall building contains cantilever floor which is considered as heavy overhang.

Table 11: Short column presence with respect to number of stories.

Number of Stories	Yes	No	Total
1	1	5	6
2	2	6	8
3	5	11	16
4	3	0	3
5	0	1	1
Total	11	23	34

In the tire 1 survey, performance score calculated for each building. Figure 5 shows the performance score obtained for RC buildings. The blue and green lines are showing a margin for the performance class. The building having a score above 75 can be classified as low risk building. The building having a score below 50 considered as high risk buildings. The score range from 50 to 75 marked as moderate risk class. Table 12 represents level 1 performance score variations with different number of stories.

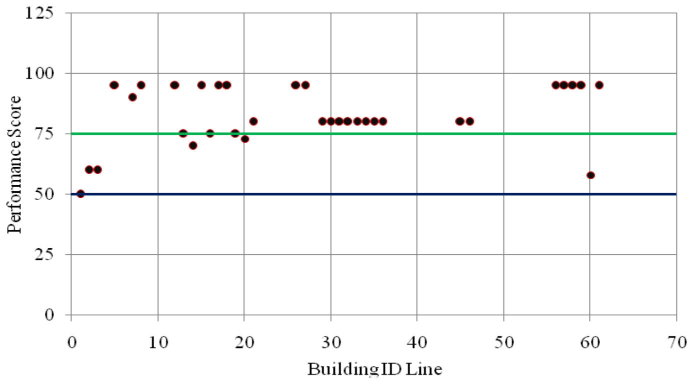


Figure 5: Performance Score obtained from walkdown survey.

Table 12: Performance Score variations with number of stories.

Number of Stories	50	51-75	> 75	Total
1	0	0	6	6
2	0	0	8	8
3	0	5	11	16
4	0	3	0	3
5	1	0	0	1
Total	1	8	25	34

RVS conducted for masonry structures only. The building with a score greater than 1.0 can be considered as low risk. However, there has no specified classification prescribed as per FEMA. It has been observed that each of these masonry buildings has same probability of collapse because of the similarity in buildings type and configaration among the buildings.

3.2 Level 2 Assessment

Second level assessment conducted for the RC buildings following Turkish Tire 2 guideline prepared by Ozcebe et al. in 2006. Eight buildings were analyzed based on building importance level in terms of building use. Academic and administrative buildings were preferred in this stage. Two buildings (identity number 01 and 16) were assumed that they are separated in two segments from their foundation. So, each of these two buildings are considered as two separate structure. Table 13 represents the summary of calculation values in level 2 assessment. These building integrity values were checked after taking detail structural floor sketch and preliminary assessment calculation steps prescribed in the methodology chapter. Figure 6 shows a typical ground floor sketch during the level 2 survey. All of the buildings (except 5 storied hall) are low risk class. The hall building falls into moderate risk group. GIS maps are shown in Figure 7 and Figure 8, where buildings location with number of stories, occupancy types, structural type and peformace scores are presented.

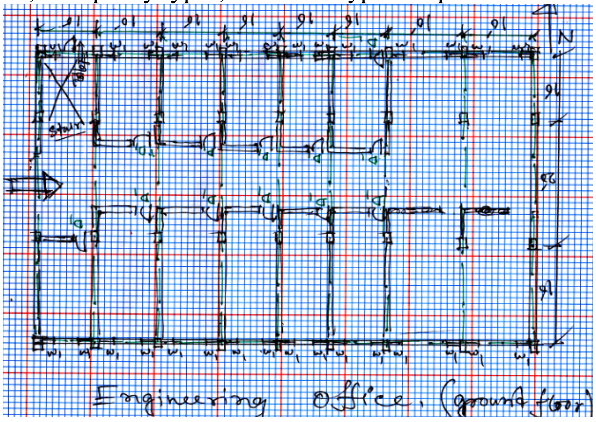
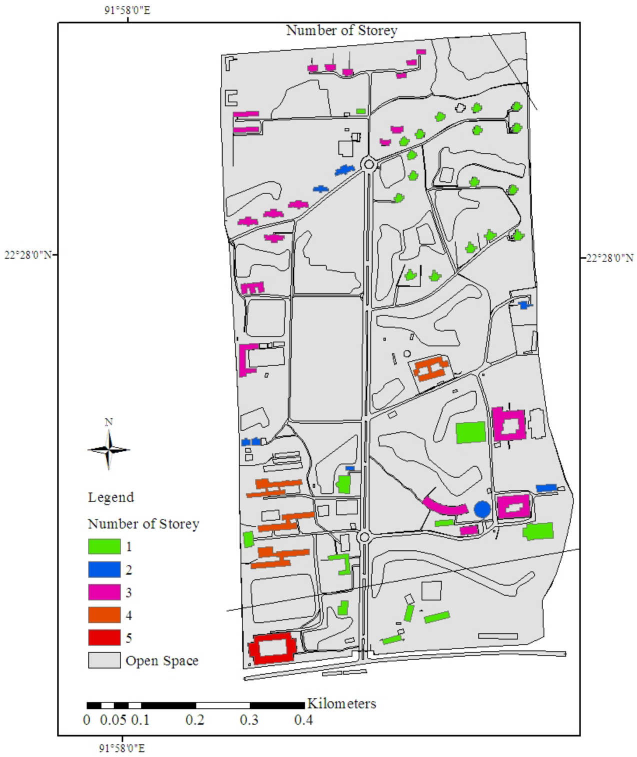
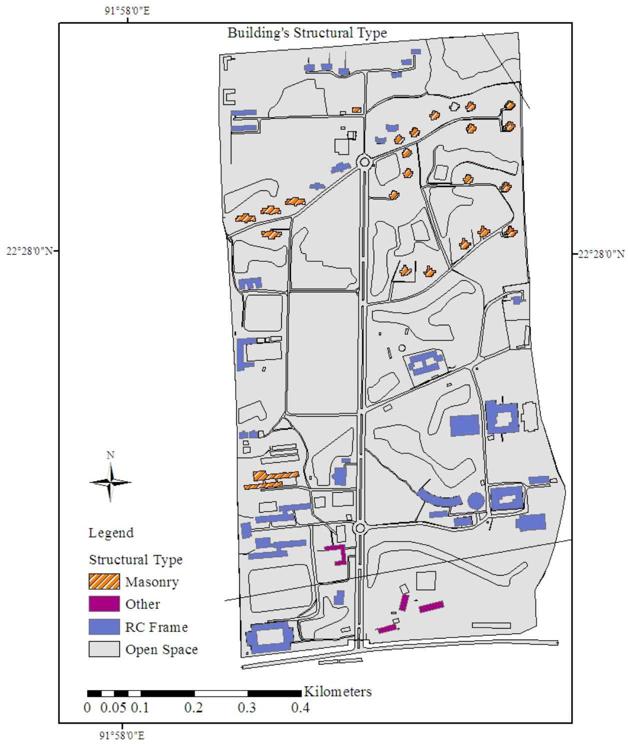


Figure 6: A ground floor sketch of a building.

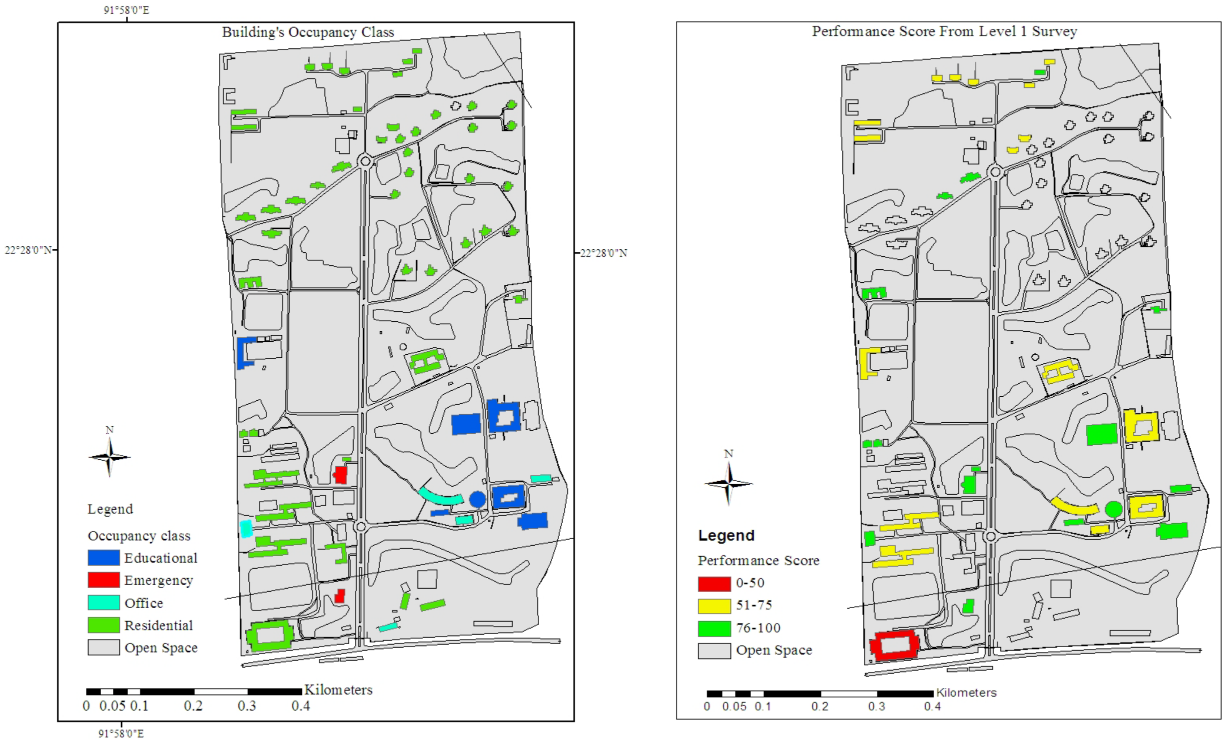


a) number of stories



b) structural types of the buildings

Figure 7: Number of stories and structural types in GIS.



a) building occupancy types b) performance socores for RC buildings

Figure 8: Occupancy types and Peformance scores in GIS.

Table 13: Summary of assessment results in tire 2.

ID No.	n	mnlstfi	mnlsi	nrs	ssi	or	DI_{LS}	DI_{IO}	CV_{LS}	CV_{IO}
C-1a	5	0.051	0.96	3	1.00	0.30	-0.0084	0.6551	1.5726	0.0012
C-1b	4	0.015	1.16	3	1.00	0.00	-1.4674	-1.3179	1.3661	0.7167
C-05	1	0.159	16.21	3	0.00	0.00	-9.3712	-5.9085	1.2167	0.5002
C-08	2	0.028	6.64	3	1.00	0.00	-3.7076	-3.5244	1.2167	0.5002
C-13	3	0.044	2.73	3	0.89	0.00	-2.7218	-2.3568	1.2167	0.5002
C-15	2	0.008	1.73	1	0.86	0.00	-1.8387	-1.6853	1.2167	0.5002
C-16a	3	0.098	3.10	3	1.00	0.00	-2.4606	-2.361	1.2167	0.5002
C-16b	3	0.109	3.50	3	1.00	0.00	-2.5359	-2.4075	1.2167	0.5002
C-18	2	0.045	3.14	3	1.00	0.00	-3.0751	-3.1556	1.2168	0.5002
C-20	3	0.017	2.76	3	1.00	0.00	-2.3785	-2.2973	1.2168	0.5002

4. CONCLUSION

This study presents a seismic vulnerability assessment application on a small scale for the CUET campus area. RVS and Turkish method are Rapid Screening Procedure to remark a conclusion before starting any detail structural assessment. The RSP is the decisive indicator that further detail structural assessment will be taken place or not. Turkish vulnerability assessment method is resonably acceptable because of the structural pattern is very similar with Bangladeshi buildings. Form the study a rapid screening database prepared which will be very useful before starting any future work at CUET. The Walkdown survey yielded the complete inventory of building stock in CUET campus. At the end of this survey it has been obtained that CUET campus contains mainly two structural types of buildings. Most of the buildings obtained good performance score from level 1 assessment. Among the surveyed buildings, only 2 percent of buildings fall into highly vulnerable to earthquake.

RVS results are very similar for all masonry buildings. This is because all of these type buildings are constricted following a unique pattern in their elevation and plan shape.Masonry buildings need to calculate in detail level for more reliable risk identification. It has been observed building performance score decreases with increasing in number of stories.

REFERENCE

BNBC (1993), *Bangladesh National Building Code (BNBC 1993),* Housing and Building Research Institute.

CDMP. (2009), *Seismic Hazard & Vulnerability Assessment of Dhaka, Chittagong & Sylhet City Corporations*, Comprehensive Disaster Management Program, Bangladesh

Federal Emergency Management Agency, (1988), *FEMA 154: Rapid Visual Screening of Buildings for Potential Seismic Hazards - A Handbook*, Washington D.C.

GSHAP (1992), *World Seismic Map*, Global Seismic Hazard Assessment Program, www.seismo.ethz.ch/GSHAP/.

Ozcebe, G., Sucuoglu, H., Yücemen, S.M., Yakut, A. and Kubin, J. (2006), *Seismic Risk Assessment of Existing Building Stock In Istanbul a Pilot Application in Zeytinburnu District*, 8NCEE, Paper No. 1737, San Francisco.

T.M. Al-Hussaini, T.R. Hossai,, and M. N. Al-Noman (2012), *Proposed Changes to the Geotechnical Earthquake Engineering Provisions of the Bangladesh National Building Code*, Geotechnical Engineering Journal of the SEAGS & AGSSEA Vol. 43, No. 2

Wu, Y. M., Teng, T. L., Shin, T. C., and Hsiao, N. C. (2003), *Relationship between Peak Ground Acceleration, Peak Ground Velocity, and Intensity in Taiwan*, Bulletin of the Seismological Society of America, Vol. 93, No. 1, pp. 386–396.

CRITICAL INVESTIGATION OF THE FINITE ELEMENT MODELS OF STEEL FIBER REINFORCED CONCRETE (SFRC): EVALUATION OF THE GOVERNING PARAMETERS TO PREDICT THE FLEXURAL CAPACITIES

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ABSTRACT

In recent years, brittle failures of so many reinforced concrete structures are observed during earthquakes and other deadly forces. In this regard, concrete researchers are intended to improve the ductility of the RC structure by using steel fiber to make concrete a composite material with enhanced capacities. To this end, this research concentrates on the applicability of steel fiber reinforced concrete (SFRC) in construction industry of Bangladesh by providing validated Finite Element (FE) models of experimental results. Experimental investigations show promising outcomes by using steel fibers in flexural specimens. Results of plain concrete and steel fiber reinforced concrete (SFRC) beam specimens are compared. SFRC beam specimens showed an increase of about 8% to 60% flexural capacity enhancement. Flexural specimens are then modeled in the Finite Element (FE) platform of ANSYS 10.0. Material property and boundary condition are applied on the basis of experimental data and test condition. Satisfactory agreement is observed between the test results and FE models. Critical investigations are done by evaluating different controlling parameters like Poisson's ratio, tensile strength, modulus of elasticity, shear transfer coefficients for open or close cracks etc. Tensile strength is one of the major governing parameters among all of those considered and it depends greatly on steel fiber aspect ratio and percentage volume of fiber. This study provides information on the parameter used in the FE model to get a realistic SFRC model which can be used to estimate flexural capacity enhancement of real structures made of SFRC.

Keywords: Steel fiber reinforced concrete (SFRC), ANSYS, Finite Element (FE) modeling and analysis, flexural capacity, aspect ratio of fiber, steel fiber volume ratio.

1. INTRODUCTION

Strength and ductility of reinforced concrete structures depend mainly on proper detailing of the reinforcement in flexural members. Brittle failure of these members may lead to catastrophic damages to the structure and the people living on these structures. Recent earthquake in different parts of the world have revealed again the importance of design of reinforced concrete with high ductility. Hughes (1997) stated that a major advantage of fiber reinforcement is their ability to absorb energy and control cracking so as to transform an essentially brittle cement-based matrix into a ductile material. To increase the ductility of the flexural members, using Steel Fiber Reinforced Concrete (SFRC) can be an efficient technique. Steel Fiber is used as a volume percentage in the concrete mortar which will create a matrix and the bond strengths increased. Load carrying capacity of steel fiber at post-cracking stage made it point of interest in modern research. According to Ghalib (1980), fiber reinforcement presents several advantages such as superior crack control, ductility, energy absorption capacity and improving the internal tensile strength of the concrete due to bonding force between the fiber and the matrix. Fiber reinforcement considerably improves the flexural strength, direct tensile strength, fatigue strength, shear and torsional strength, shock resistance, ductility and failure toughness of concrete (Uddin et al., 2013). For many years, ACI 544.4R-88 has been working towards the development of standardized testing techniques as applied to fiber reinforced concrete. The committee suggested that the work is not finished and a continuous research effort is needed to improve testing and reporting methods for SFRC. Based on past study, experimental investigations are conducted to study the increase in flexural capacities of simply supported, cantilever and fixed beam using steel fiber reinforced concrete (SFRC).

A Finite Element Analysis (FEA) software package ANSYS 10.0 is used to analyze the flexural elements and introduce a good concrete model for Steel Fiber Reinforced Concrete (SFRC) as well as plain concrete made of brick and stone aggregate. Finite Element (FE) Modeling and Analysis is being preferred as a widely accepted method to study the behavior of concrete. The objective of the FE Modeling and Analysis is to verify the results with the experimental measurements conducted in the present research and to propose an acceptable SFRC model to be applied in further analysis. All the flexural specimens of this investigation are modeled in ANSYS 10.0 platform. Two different Poison's ratios for brick and stone concrete are selected by comparing FE output with the stress-strain behavior in tension and compression from experiments. A reasonable modeling of concrete on a finite element (FE) platform using suitable element type, adequate mesh size, appropriate boundary conditions, realistic loading environment and proper time stepping can help to estimate the governing parameters of concrete. Using these governing parameters (i.e. Poison's ratio, tensile strength, and the stress-strain relationship) all the flexural members (simply supported, cantilever, and fixed supported beams of plain concrete and SFRC) are modeled, analyzed and compared the results gathered from experimental outcomes. After evaluation of this parameter by extensive analysis, Finite Element (FE) models showed a good correlation with the experimental results and also showed similar failure patterns. This investigation is intended to validate the FE models of flexural members with the experimental results by identifying and using the pertinent parameters of the concrete model as well as to provide a successful FE SFRC model for analyzing future problems on SFRC.

The main objective of this research is to model the steel fiber reinforced concrete (SFRC) in the finite element platform and predicting the property enhancements by validating FE outputs with experimental flexural capacities. Besides these, this investigation also tends to examine failure patterns of beams made of SFRC. In the absence of a reliable model for predicting the flexural strength (with different support condition) of SFRC with the easily available fibers in context of Bangladesh, the current research aims to investigate the capacity enhancement and stress field of the SFRC from experimental and numerical viewpoint to introduce this new engineering material in the construction industries of Bangladesh.

2. MECHANISM OF CRACK ARRESTING IN SFRC

In reinforced concrete structures, cracks are initiated due to the tension and propagate to the neutral axis of the flexural member. Steel fibers arrest the micro-cracks and cracks are resisted by the tensile property of the fiber. In the hardened state, when fibers are properly bonded, they interact with the matrix at the level of micro-cracks and effectively bridge these cracks thereby providing stress transfer media that delays their coalescence and unstable growth (Figure 1). If the fiber volume fraction is sufficiently high, this may result in an increase in the tensile strength of the matrix. Indeed, for some high volume fraction fiber composite, a notable increase in the tensile/flexural strength over and above the plain matrix has been reported. Once the tensile capacity of the composite is reached, and coalescence and conversion of micro-cracks to macro-cracks has occurred, fibers, depending on their length and bonding characteristics continue to restrain crack opening and crack growth by effectively bridging across macro-cracks. This post peak macro-crack bridging is the primary reinforcement mechanisms in majority of commercial fiber reinforced concrete composites.

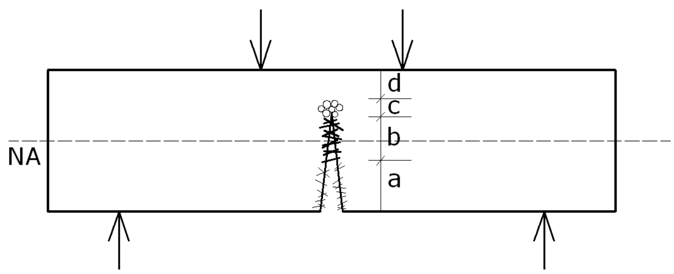
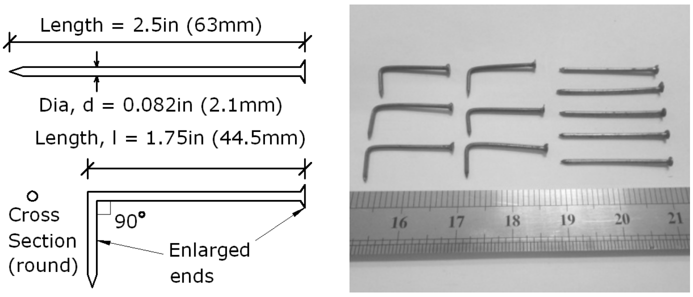


Figure 1: Mechanism of fibers in flexure (a) free area of stress, (b) fiber bridging area, (c) micro-crack area and (d) undamaged area.

3. EXPERIMENTAL PROGRAM

The fiber volume is taken 1.5% (represented as SF1.5 in specimen designation) to cast the flexural specimens. The fibers are enlarged end fibers which is easily available in the market. The fibers are customized to make enlarged ends for better anchorage (Figure 2a and b). Two different aggregate types are used to make SFRC and plain concrete specimens, i.e. stone (CS) and brick (CB) aggregate and the effects of SFRC on these two types of concretes are evaluated. Stone and brick concretes show substantial increase the flexural capacity and ductility due to the presence of steel fibers. Three kinds of flexural capacity are tested, i.e. cantilever (FC), fixed (FF) and simply supported (FS). Concretes are made of OPC i.e. Ordinary Portland cement (O). All the specimens are tested in a 1000kN capacity digital universal testing machine (UTM). Strain data are measured by applying digital image correlation technique (DICT) using high definition (HD) images and high speed video clips and these data are synthesized with the load data from the load cell of UTM which is also followed in the work of Islam (2011), Islam et al. (2011), Uddin et al. (2013) and Dola et al. (2013). Flexural test shows the increase in flexural capacity of about 8% to 60% of beams due to SFRC which also showed an indication of increase in ductility of the flexural member. All the specimens are then modeled in the FE platform of ANSYS 10.0 and also validated with the experimental results and failure patterns.



(a) (b)

Figure: 2: (a) and (b) Size and geometry of the customized enlarged ends steel fibers employed in this research.

4. FINITE ELEMENT MODELING AND ANALYSIS

4.1 Element type

All the flexural specimens are modeled on the FE platform of ANSYS 10.0. An eight node solid element, SOLID65 is used to model the concrete and also SFRC. The solid has eight node with three degrees of freedom at each node-translational in the nodal x, y, and z directions. The element is capable of plastic deformation, cracking in three orthogonal directions and crushing. In concrete applications, for example, the solid capability of the element is used to model the concrete while the rebar capability is available for modeling reinforcement behavior. Other cases for which the element is also applicable would be reinforced composites (ANSYS 2005), such as, fiberglass as well as fiber reinforced concrete (FRC). The geometry and node locations for SOLID65 element are shown in Figure 3.

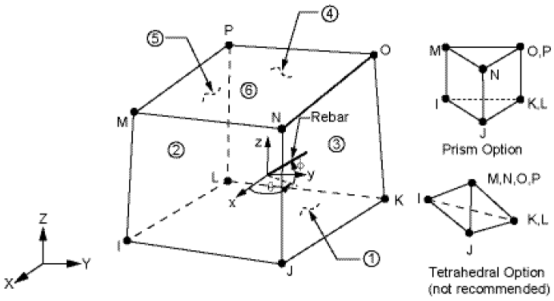


Figure 3: Geometry of SOLID 65 in ANSYS 10.0 platform

4.2 Material properties concrete

Development of model for the behavior of concrete is a challenging task. Concrete is a quasi-brittle material and has different behavior in tension and compression. The tensile strength of concrete is typically 8-15% of the compressive strength (Shah et. al., 1995). In compression, the stress-strain curve for concrete is linearly elastic up to 30 percent of the maximum compressive strength. Above this point, the stress increases gradually up to the maximum compressive strength. After it reaches the maximum compressive strength, the curve descends into a softening region, and eventually crushing failure occurs at an ultimate stain. In tension, the stress-strain curve (Figure 4) for concrete is approximately linearly elastic up to the maximum tensile strength. After this point, the concrete cracks and strength decreases gradually to zero (Bangash, 1989).

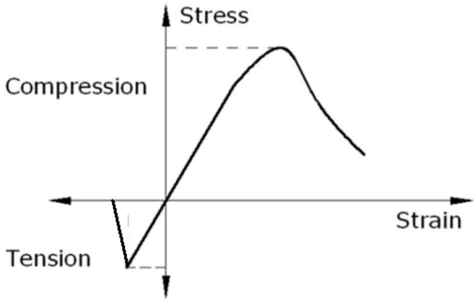
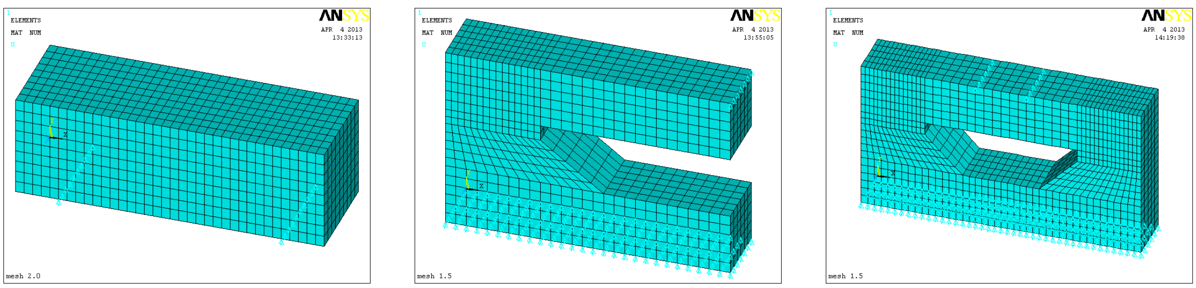


Figure 4: Typical uniaxial compressive and tensile stress-strain curve for concrete

To model the concrete and SFRC perfectly, ANSYS requires to provide data for material properties such as (i) elastic modulus, (ii) ultimate uniaxial compressive strength, (iii) ultimate uniaxial tensile strength and (iv) Poisson's ratio. All the values are provided from experimental outputs. Poisson's ratio for concrete and SFRC is estimated to be 0.25 and 0.35 for stone and brick concretes respectively by extensive numerical trials and matching experimental data. William and Warnke (1975) failure criterion is applied to model the concrete as well as SFRC. Four important parameters, i.e. i) shear transfer coefficients for an open crack, ii) shear transfer coefficients for a closed crack, iii) uniaxial tensile cracking stress and iv) uniaxial crushing stress are also considered to model the concretes. Typical shear transfer coefficients range from 0.0 to 1.0, with 0.0 representing a smooth crack (complete loss of shear transfer) and 1.0 representing a rough crack (no loss of shear transfer). The shear transfer coefficients for open and closed cracks are determined from the work of Kachlakev, et al. (2001) as a basis. Convergence problems occurred when the shear transfer coefficient for the open crack dropped below 0.2. No deviation of the response occurs with the change of the coefficient. Therefore, the coefficient for the open and close crack is set to 0.3 and 1.0 for SFRC.

4.3 Geometry of FE specimens

The dimension of the simply supported beam specimen is 6x6x24 in (153x153x610 mm) and is analyzed with two point loading and each load was placed at a distance of 6in (153 mm) from the end supports. Loading is applied as displacement boundary condition. Support was placed 3 in (76 mm) from the edge of the beam. The span between the two supports was 18 in (458mm). These beams are restrained only vertically as it was tested experimentally. The dimension of fixed supported beam specimens is 4x4x12in (100x100x300mm) and is connected monolithically with a relatively higher stiffened part, which is used to act as fixed support. The beam is loaded by two points loading as displacement boundary condition and supports are applied at the bottom nodes restricting into only vertical movement. The dimension of cantilever beam specimen is 4x4x12in (100x100x300mm). The cantilever portion is monolithically attached to a relatively higher stiffened part which is acted as a fixed support during the loading. An inclined part is extended in order to resist the bending of the lower vertical part and to get the maximum resistance against rotation. Universal Testing Machine (UTM) is used for the experiment where the specimen is placed on a flat surface and loaded on the cantilever portion at the end. The specimen is supported at the bottom nodes by restricting the movement on the vertical direction. Figure 5 shows typical diagram of FE models with boundary conditions.



(a) (b) (c)

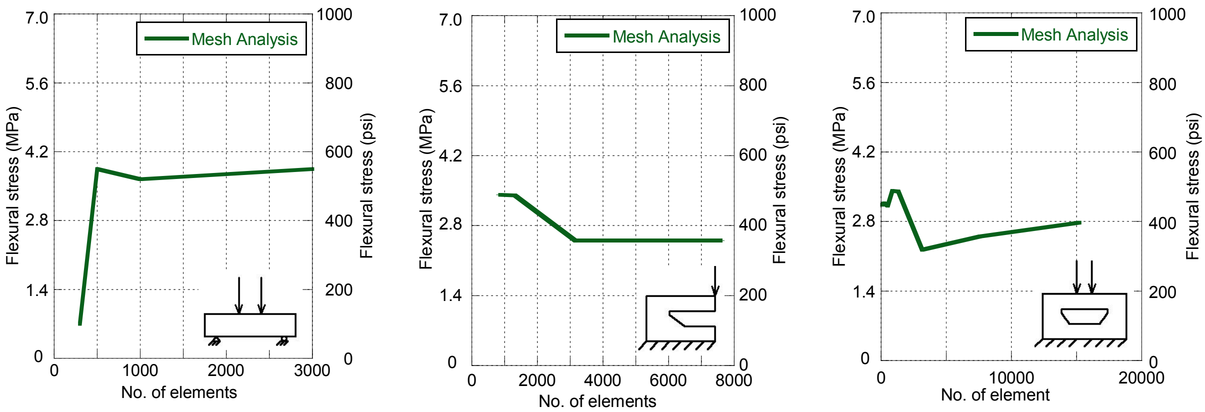
Figure 5: Boundary condition of (a) simply supported, (b) cantilever and (c) fixed supported specimens.

4.4 Loading and boundary conditions

Experimentally all the loading are applied as displacement by the Universal Testing Machine (UTM) at a rate of 0.05 in. (1.27mm) per min. As a result, displacement boundary conditions are enforced to constrain the model to get a unique solution. The boundary condition used in the experimental tests are applied but by avoiding geometric instability. Figure 5 shows boundary condition of simply supported, cantilever and fixed supported beam. The displacement boundary condition is applied in 500 steps followed by 2 sub-steps for each step.

4.5 Mesh size analysis

Mesh size plays an important role in Finite Element Analysis. A suitable mesh size is to be chosen to achieve sufficient accuracy and at the same time not to extend the run time too long. Mesh size may vary in the analysis of a single structure. Different mesh alignments generally give slightly varying solutions. In fact in real life problems mesh size is constantly refined to get a representative solution. To get the accurate result number of element vs. stress graph is plotted and found interesting relationship between them. Element stress changes with the increase in number of element of a specimen, but at a certain ratio stress increasing curve become horizontal. It means after a certain number of element stress will not vary significantly. The numbers of elements at the horizontal location of stress vs. number of elements curves are considered during FE meshing of a particular specimen.



(a) (b) (c)

Figure 6: Stress vs. number of elements relationship for (a) simply supported beam, (b) cantilever beam and (c) fixed supported beam.

Figure 6 shows that at a low number of elements, stress is changing abruptly whereas at a higher number of element reaction became stable, but so much higher number of element will increase the complexicity and also the analysis time of the specimen. A reasonable number of elements, is selected with maximum accuracy and minimum analysis time. The number of elements taken for the FE analyses are 2000, 8000 and 12000 for the simply supported, cantilever and fixed supported beam specimens in ANSYS.

5. VALIDATION OF FINITE ELEMENT MODELS

5.1 Validation of flexural capacity

Figures 7 & 8 show the validations of flexural stress results gathered from the experimental measurements with the FE analysis by ANSYS 10.0. It satisfactorily demonstrates the accuracy of the FE model of plain concrete as well as SFRC made of brick and stone concrete respectively. The flexural stresses of cantilever specimens are taken at the root of the cantilever, at the midpoints for the simply supported and fixed supported specimens. The FE results in most of the cases found to be more or less conservative with respect to the experimental outcomes which also ensure higher factor of safety as well as reliability of the models. In all the cases the increase of flexural capacity is found which is also seen in the experimental results. In most of the cases the ANSYS curve is following the experimental path but in a conservative fashion which indicate the higher factor of safety of these FE models. The accuracy of the prediction and adjustment of the governing parameters helped to validate the models. Even for the new specimens i.e. cantilever and fixed supported beam specimens, the stress paths and failure patterns were completely unknown. After the FE analysis, similar results and failure patterns are found which enhanced the confidence of the validity of the modeling and analyses. Cantilever specimens made of brick and stone concrete are found to be similar stress path (Figure 7a, 7b, 8a and 8b). This is also found true for the fixed and simply supported beams.

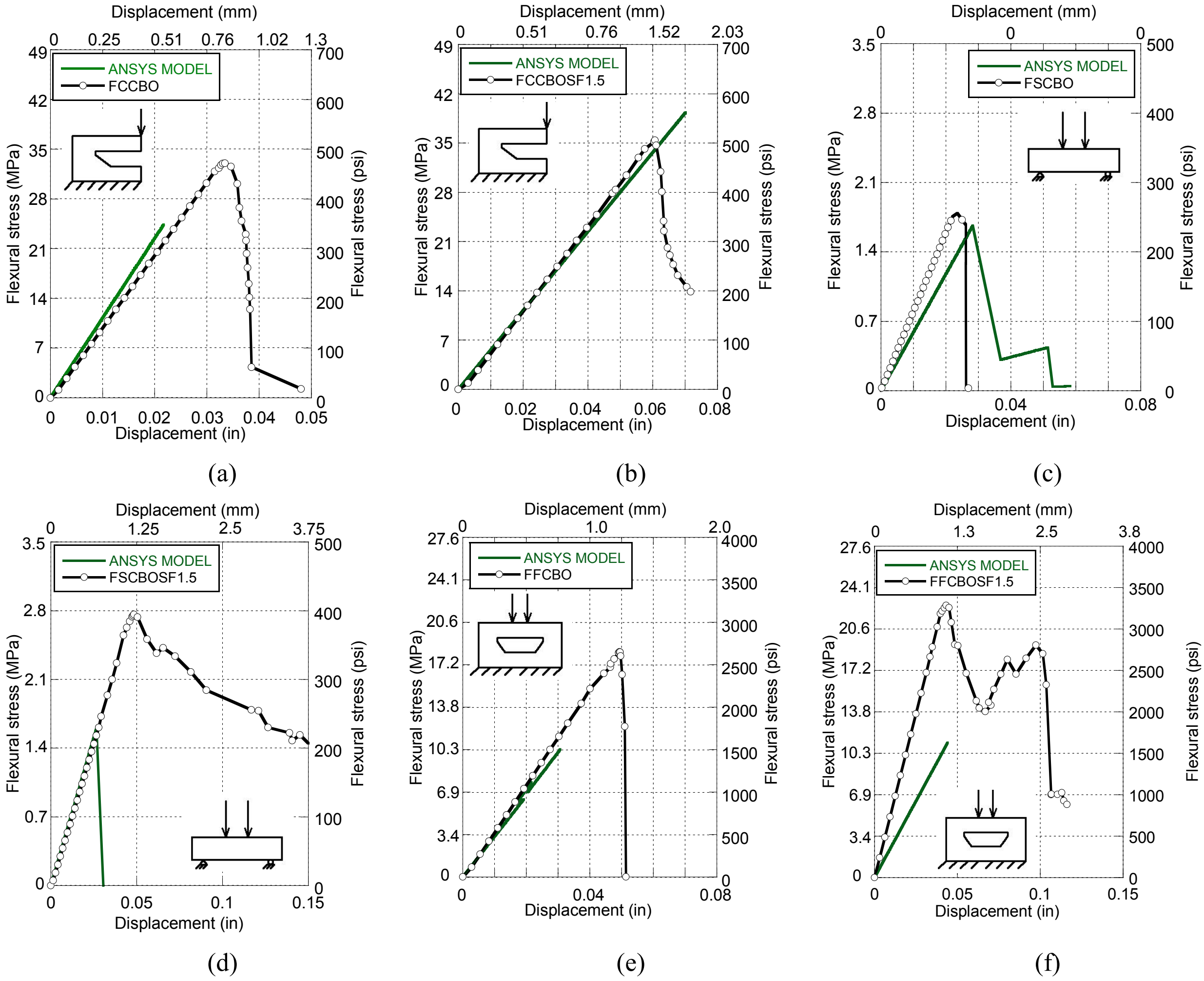
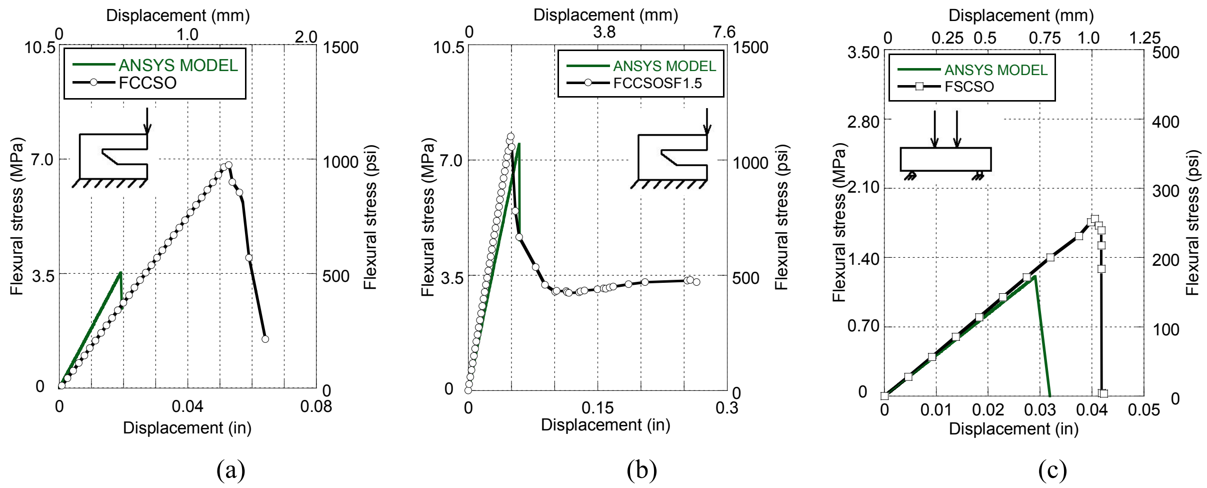
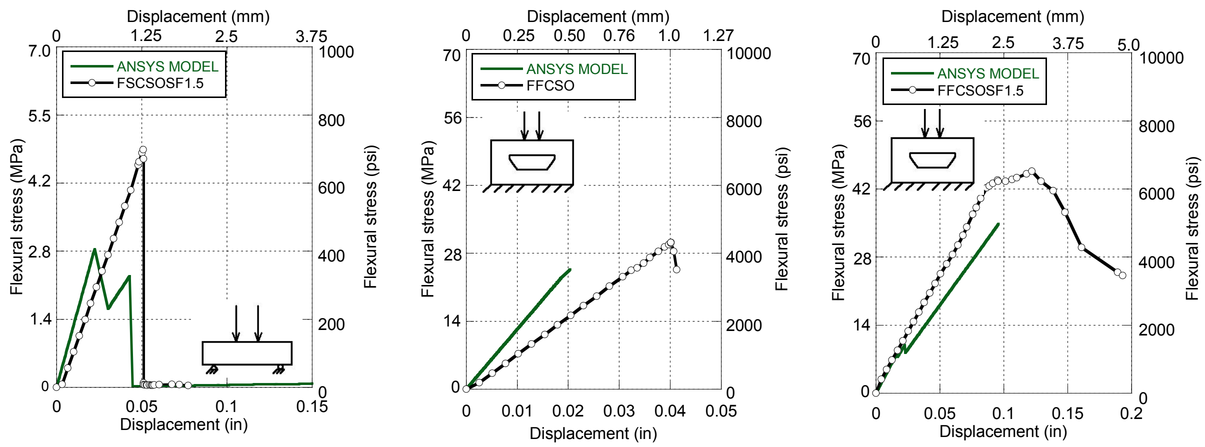


Figure 7: Validation of FE model of plain concrete and brick SFRC.





(d) (e) (f)

Figure 8: (a) to (f) Validation of FE model of plain concrete and brick SFRC.

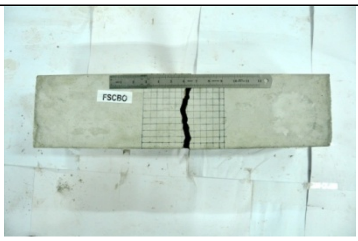
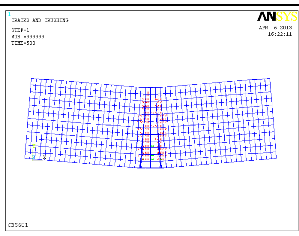
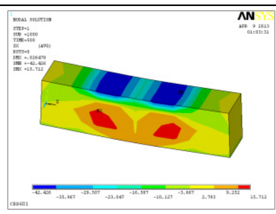
5.2 Validation of failure pattern

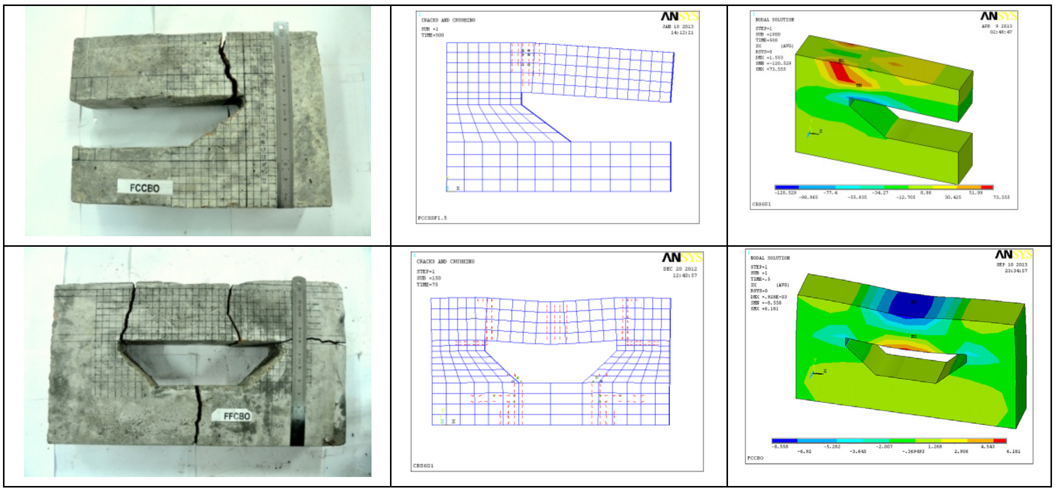
All failure patterns of plain and SFRC brick and stone concrete specimens are observed and analyzed. Fixed support condition concept is totally new for this kind of investigation, so crack formation and failure pattern looked unusual at first sight, but later from the Finite Element analysis, it is found logical to form this type of crack and is validated. Most of the cracks are formed due to tension which is given in Table 1. Table 1 shows the similarities of failure pattern between the experimental specimen and the FE model using ANSYS. This indicates that the FE modeling of SFRC beam specimens using the pertinent parameters gathered from experimental testing are validated and there remains a good agreement as well as it can be used in future SFRC model of different fiber volume, span lengths and support conditions.

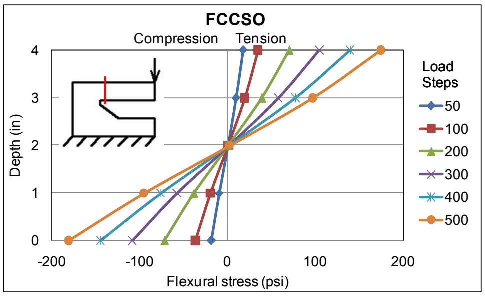
5.3 Critical investigation via FE analyses

After the validation of FE models, it becomes easier to observe the stress distribution with respect to depth in the concrete beams. It can be said that the validated FE models are representing almost true picture of the experimental specimens. Some critical investigations on the stress distribution with load increment for stone concrete specimens are shown in Figures 9, 10 and 11. Figure 9 shows the stress distribution of cantilever specimens (red line representing the critical sections). Tension occurred at top and compression at the bottom and both the tensile and compressive stresses i.e. flexural stresses are found much higher in SFRC specimens. Also the neutral axis (NA) is found slightly lower in the SFRC specimen which is showing shifted slightly to the tension zone. This is also found true in case of the fixed supported specimens (Figure 10). The tensile stress is found higher in the SFRC fixed supported specimens and during higher load steps the compression zone found failed releasing the stress while tensile stress remained high enough. For the simply supported specimens the compressive and tensile stresses are quite low but for SFRC the flexural stresses found much higher compared to plain concrete which again represents the logical behaviour.

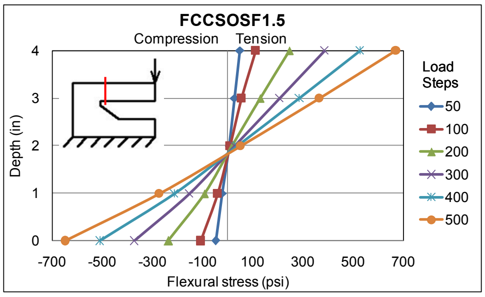
Table 1: Evaluation of failure pattern

Failed experimental specimen	**Failed FE model in ANSYS**	**Flexural stress contour**
		



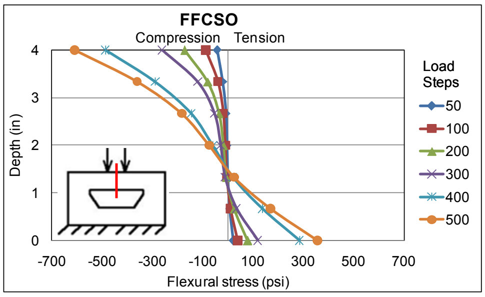


(a)

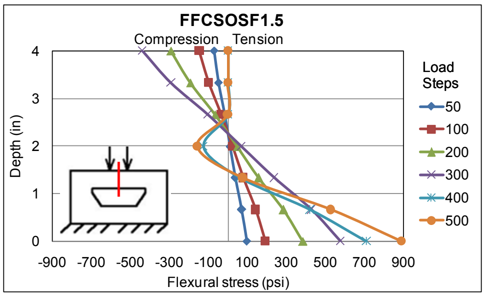


(b)

Figure 9: Stress distribution of cantilever specimen at the root of cantilever by ANSYS.

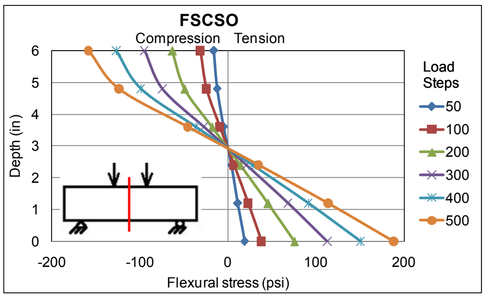


(a)

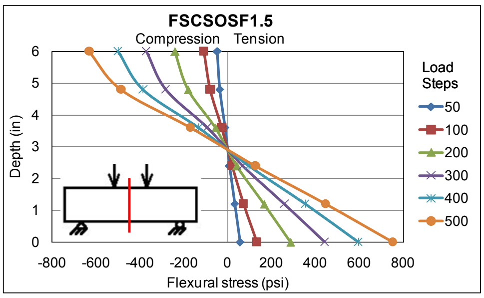


(b)

Figure 10: Stress distribution of fixed supported specimen by ANSYS.



(a)



(b)

Figure 11: Stress distribution of simply supported specimen by ANSYS.

6. CONCLUSIONS

The following conclusions can be drawn from this investigation:

- Flexural test shows the increase in flexural capacity of about 8% to 60% of beams due to SFRC which also showed an indication of increase in ductility of the flexural member.
- The FE showed similar results which ensures the validity of the models and the FE models are successfully capable of predicting the enhanced capacities due to SFRC.
- The failure patterns and locations of FE models are wonderfully matched with the experimental results which also validate the FE modeling.
- The FE analyses showed conservative results which ensure reliability of the FE models.
- The stress distribution evaluation from FE analyses provides information of critical location of the experimental specimens.
- These FE modeling will definitely provide invaluable information of this engineering material to the construction industry of Bangladesh.

REFERENCES

ACI 544.4R-88 (1999). "Design Considerations for Steel Fiber Reinforced Concrete", American Concrete Institute, Reported by ACI Committee 544.

ANSYS Structural Analysis Guide (2005), Version 10.0, ANSYS, Inc., Canonsburg, Pennsylvania.

Bangash, M.Y.H. (1989). "Concrete and concrete structures: Numerical Modeling and Applications", Elsevier Science Publishers Ltd., London, England.

Dola, J. F., Islam, M. R., Khatun, M. S. and Hussan, M. (2013). "Investigation of the shear capacity of beams made of steel fiber reinforced concrete (SFRC)", B.Sc. Engineering Thesis, Department of Civil Engineering, Ahsanullah University of Science and Technology, Dhaka, April.

Ghalib, M.A. (1980). "Moment Capacity of Steel Fiber Reinforced Small Concrete Slabs", ACI Journal, Title no: 77-27, July- August.

Hughes, B.P. (1981). "Design of Prestressed Fiber Reinforced Concrete Beams for Impact", ACI Journal, Title no: 78-25, July-August.

Islam, M.M., Choudhury, M.S.I., Abdulla, M. & Amin, A.F.M.S. (2011). "Confinement Effect of Fiber Reinforced Polymer Wraps in Circular and Square Concrete Columns", 4th Annual Paper Meet and 1st Civil Engineering Congress, Civil Engineering Division, Institution of Engineers, Bangladesh (IEB), 22-24 December, Dhaka (Awarded best paper).

Islam, M.M. (2011), "Interaction diagrams of square concrete columns confined with fiber reinforced polymer wraps", M.Sc. Engineering Thesis, Department of Civil Engineering, Bangladesh University of Engineering and Technology (BUET), Dhaka, August.

Kachlakev, D.I., Miller, T.Y., Chansawat, S. and Potisuk, T.K. (2001). "Finite Element Modeling of Reinforced Concrete Structures Strengthened With FRP Laminates", California Polytechnic State University, San Luis Obispo, CA and Oregon State University, Corvallis, OR for Oregon Department of Transportation, May.

Shah, S.P., Swartz, S.E. and Ouyang, C. (1995). "Fracture Mechanics of Concrete", John Willy & Sons, Inc., New York.

Uddin, M.K., Mustafiz, A. Z., Chowdhury, M. A. and Mondal, P. (2013). "Investigation of the Flexural Capacity of Beams made of Steel Fiber Reinforced Concrete (SFRC)", B.Sc. Engineering Thesis, Department of Civil Engineering, Ahsanullah University of Science and Technology, Dhaka, April.

Willam, K.J. and Warnke, E.D. (1975). "Constitutive Model for the Triaxial Behavior of Concrete", Proceedings, International Association for Bridge and Structural Engineering, ISMES, Bergamo, Italy, Vol. 19, pp 1-30, 1975.

FINITE ELEMENT MODELING AND ANALYSIS OF RC BEAMS MADE OF STEEL FIBER REINFORCED CONCRETE (SFRC): CRITICAL INVESTIGATION OF THE FLEXURAL AND SHEAR CAPACITY ENHANCEMENTS

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ABSTRACT

Steel fiber reinforced concrete (SFRC) is being used in the construction industries for decades for their improved flexural strength, direct tensile strength, fatigue strength, shear and torsional strength, shock resistance, ductility and failure toughness. In Bangladesh, it is not yet well introduced in the construction industry due to lack of reliable experimental results and Finite Element (FE) modeling. To this end, this research intends to provide evaluation of extensive analyses on the flexural and shear capacity enhancements of the RC specimens in ANSYS 10.0 FE platform. Poisson's ratio, tensile strength, modulus of elasticity, compressive stress-strain behaviour, shear transfer coefficients for open or close cracks of plain concrete and SFRC are taken as governing parameters to validate the FE models. The strategy to observe the improvement of flexural capacity RC members made of plain concrete and SFRC is to create the models weak in flexure by providing no flexural reinforcement but higher web reinforcements. In case of evaluating shear capacity, this scenario is made opposite. In this way the actual capacity enhancements are figured out. This study provides reliable FE models of RC members made of SFRC on the basis of experimental outputs which may help the construction industry of Bangladesh to introduce this composite material.

Keywords: Steel fiber reinforced concrete (SFRC), ductility, ANSYS, Finite Element (FE) modelling and analysis, flexural capacity, shear capacity.

1. INTRODUCTION

The use of steel fiber-reinforced concrete (SFRC) is increasing in many countries due to its improved material and structural behavior relative to plain concrete and even to conventionally reinforced concrete with the same steel volume fraction. In Bangladesh the use of steel fiber reinforced concrete has not yet been started, indeed a lot of researches are started on fiber reinforced concrete. One of the most beneficial aspects of the use of fibers in concrete structures is that non-brittle behavior after concrete cracking can be achieved with fibers (Lee et al., 2011). Adding steel fibers increases ultimate shear strength, reduces deflections, increases stiffness and transforms failure modes from brittle and dangerous shear failures into more ductile flexural failures (Yakoub, 2011). The objectives for the addition of fibers are to improve the tensile strength, flexural strength, impact strength or toughness to change the mode of failure by means of post cracking ductility to control cracking (Traina and Mansour, 1991). The main applications of steel fiber-reinforced concrete (SFRC) are in structures subjected to potentially damaging concentrated and dynamic load (Balaguru and Najm, 2004).

One of the most useful applications of SFRC would be to relieve reinforcing steel congestion by reducing the amount of shear or confining transverse reinforcement without sacrificing structural performance (Kang et al., 2011). The use of fibers would save considerable time and would facilitate placement in highly reinforced structures (Casanova and Rossi, 1997).

Steel fiber reinforcement is also effective in increasing punching shear resistance and deformation capacity of slab-column connections under combined gravity load and lateral displacement reversals (Cheng and Montesinos, 2010). In high seismic risk regions, to improve confinement, closely spaced hoops often result in highly congested columns that may cause problems during construction. The use of SFRC in such columns may permit a reduction in the amount of transverse reinforcement, leading to improved constructability (Aoude et al., 2009). For many years, ACI 544.4R-88 has been working towards the development of standardized testing techniques as applied to fiber reinforced concrete. The committee suggested that the work is not finished and a continuous research effort is needed to improve testing and reporting methods for SFRC.

In this research, RC beams made of plain concrete and SFRC are modelled in the FE platform of ANSYS 10.0 to investigate and evaluate the shear and flexural behaviour as well as stress distribution. The plain concrete and SFRC are modelled by optimizing the pertinent parameters of concrete, i.e., modulus of elasticity, Poisson's ration, compressive stress-strain relationship, density, tensile strength, shear transfer coefficient for open and close crack etc. These models will be helpful to predict the behaviour of RC structures made of SFRC which may be useful for the construction industry of Bangladesh.

2. INVESTIGATION STRATEGY

Two special types of RC beams are modelled in the FE platform of ANSYS 10.0 to observe the flexural and shear capacity enhancement due to SFRC. The sizes of the beams are 6x5x36 in (150x125x900mm) with 30in (750mm) effective span. To evaluate the flexural capacity enhancement, beams (flexure critical beam) are modelled with higher shear reinforcement and without any flexural rebar to get the actual capacity enhancements (Figure 1). Again, high flexural reinforcements are provided in beams (shear critical beams) with no web reinforcement to investigate the shear capacity enhancements (Figure 1). The beam dimensions, loading and reinforcement arrangements are showed in Figure 1.

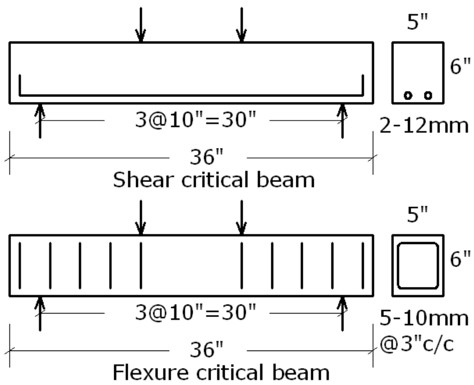


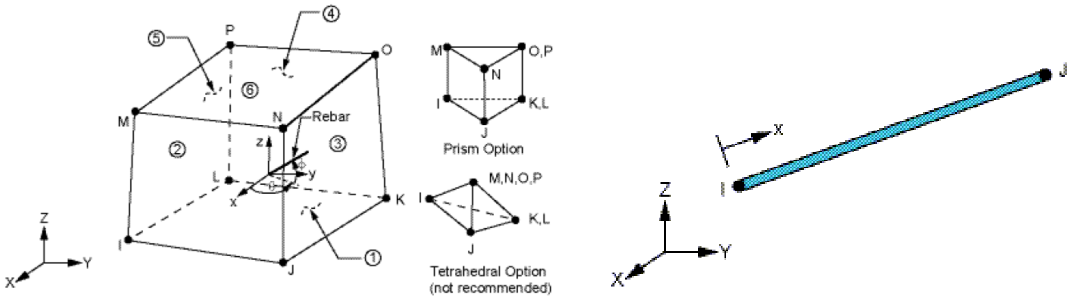
Figure 1: Beam dimensions, loading and reinforcement detail of shear and flexure critical beams.

3. MATERIAL PROPERTIES AND FE MODELING

The concrete and SFRC of are modeled using SOLID65 element in ANSYS 10.0, which is a three dimensional (3D) solid element having eight nodes with three degrees of freedom at each node, i.e., translational in the nodal x, y, and z directions. The element is capable of plastic deformation, cracking in tension and crushing in compression. In concrete applications, for example, the solid capability of the element is used to model the concrete while the rebar capability is available for modeling reinforcement behavior. Other cases for which the element is also applicable would be reinforced composites (ANSYS 2005), such as, fiberglass and in our case steel fiber reinforced concrete (SFRC). The flexural and shear reinforcements are modeled using LINK8 element, which is a 3D spar element as well as a uniaxial tension-compression element with three degrees of freedom at each node, i.e., translations in the nodal x, y, and z directions. As in a pin-jointed structure, no bending of the element is considered. This element includes of plasticity, creep, swelling, stress stiffening and large deflection capabilities. The geometries and node locations for these elements has shown in Figure 2.

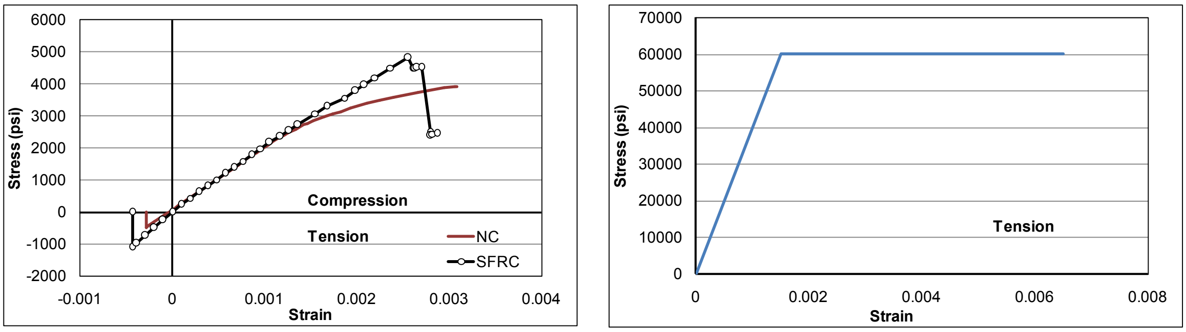
Concrete is a quasi-brittle material and has different behavior in tension and compression. In compression, the stress-strain curve for concrete is linearly elastic up to 30 percent of the maximum compressive strength. Above this point, the stress increases gradually up to the maximum compressive strength. After it reaches the maximum compressive strength, sometimes the curve descends into a softening region or eventually crushing failure occurs at an ultimate stain. In tension, the stress-strain curve (Figure 3) for concrete is approximately linearly elastic up to the maximum tensile strength. After this point, the concrete cracks and strength decreases to zero. The stress strain relationship of normal concrete (NC, considering stone concrete), steel fiber reinforced concrete (SFRC) and reinforcements used in this investigation are shown in Figure 3. The tensile strength of

SFRC is applied 1100psi (Uddin et al 2013 and Dola et al 2013) considering 1.5% volume fraction. The density of NC, SFRC and steel rebar are considered 0.086, 0.089 and 0.283 lb/cft respectively. Willam and Warnke (1975) failure criterion is applied in this modeling. The shear transfer coefficients for open and closed cracks are determined from the work of Kachlakev, et al. (2001) as a basis. Convergence problems occurred when the shear transfer coefficient for the open crack dropped below 0.2. No deviation of the response occurs with the change of the coefficient. Therefore, the coefficient for the open and close crack is set to 0.3 and 1.0 for NC and SFRC. The tensile strength is found to be the most effective controlling parameter to model NC and SFRC. The meshed beams are shown in Figure 4. In the FE modeling, the displacement boundary condition is applied as loading to provide the actual loading environment. The displacement boundary condition is applied in 500 steps followed by 2 sub-steps for each step. Figure 5 shows typical stress contours of SFRC beams.



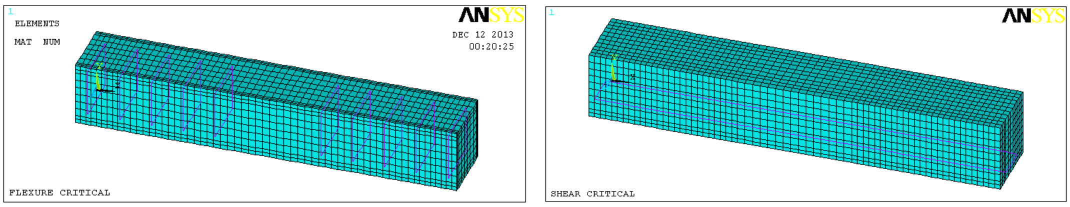
(a) (b)

Figure 2: Geometry of (a) SOLID 65 and (b) LINK8 element in ANSYS 10.0 platform.



(a) (b)

Figure 3: Stress-strain relationship of (a) NC and SFRC, (b) rebar.



(a) (b)

Figure 4: (a) Flexure critical beam and (b) shear critical beam in ANSYS 10.0

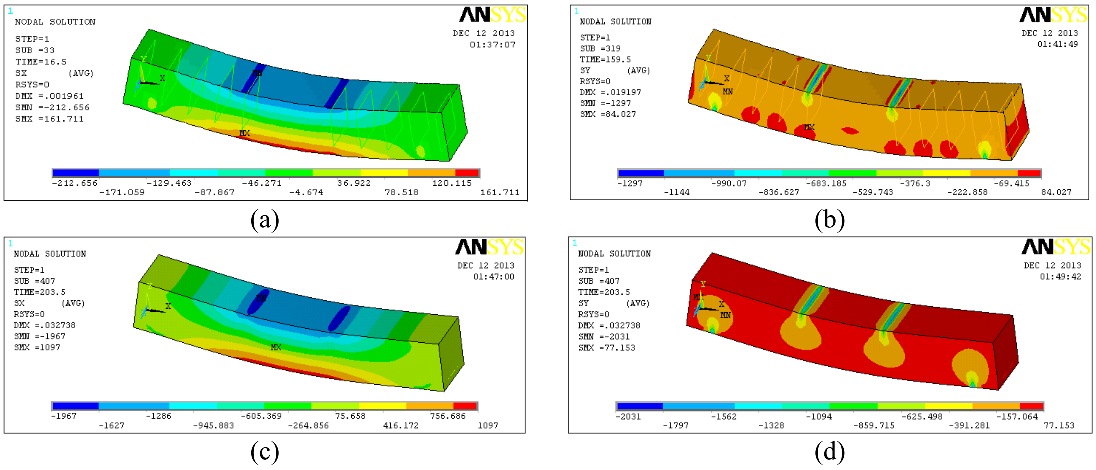


Figure 5: (a), (b) Stress contour at x and y direction respectively for flexure critical SFRC beam, and (c), (d) Stress contour at x and y direction respectively for shear critical SFRC beam.

4. OUTCOMES OF FE ANALYSES

It is clearly found the effect of steel fibers for both shear critical and flexure critical SFRC beams from their load displacement behaviour compared the corresponding NC beams in Figure 6. The load carrying capacity is found higher for SFRC beams. In case of shear critical NC beam, the beam first fails at 5500lb and took up to 6000lb, but the SFRC beam took almost 13000lb load due to steel fibers, i.e., load carrying capacity increased 217% and ductility increased 2.7 times. In case of flexure critical beams, as there are no flexure rebar, the load carrying capacity is found lower than shear critical beams. The NC beam failed at 4000lb where the SFRC beams took up to 8000lb load, i.e., load carrying capacity increased 200% and ductility increased 2.55 times. In every case the effect of steel fiber is clearly visible which ensures reliability of FE modeling.

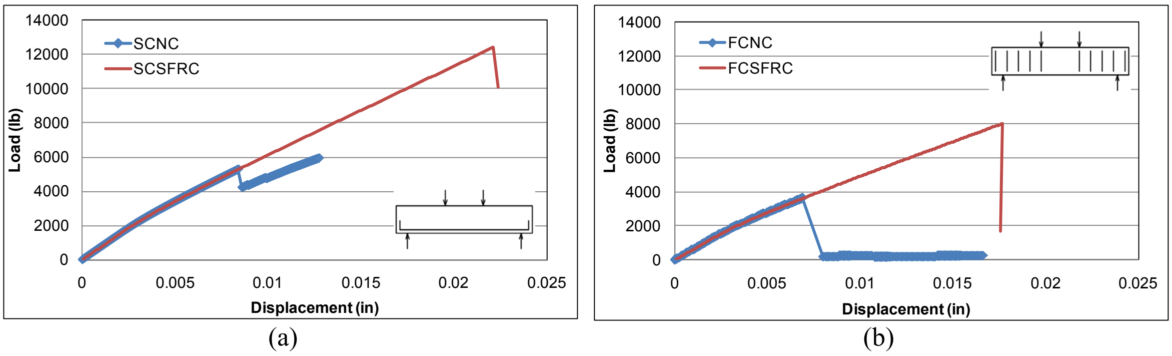
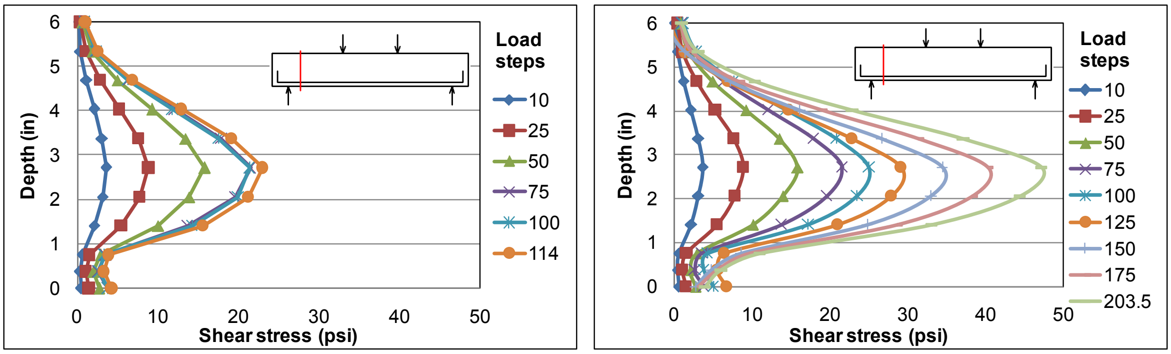


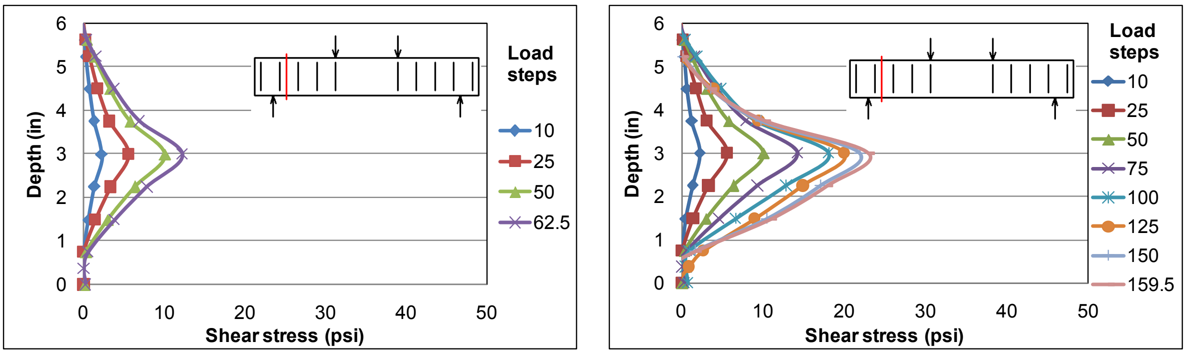
Figure 6: Load displacement behaviour of (a) shear critical normal concrete (SCNC) and shear critical SFRC (SCSFRC) beams (b) flexure critical normal concrete (FCNC) and flexure critical SFRC (FCSFRC) beams.

Figure 7(a) and 7(b) represents the shear stress distribution of shear critical NC and SFRC beams near the left support. Clearly the shear stress is found higher in SFRC beams due to its higher tensile capacity. This scenario is similar for the flexure critical beams in Figure 8(a) and 8(b). Shear stress has increased 204% in flexure critical SFRC beams and 192% in flexure critical SFRC beams. The shear stress values are higher in flexure a critical beam which is due to absent of web reinforcement. The flexural stress distribution at the mid span for the shear critical and flexure critical beams are represented in Figure 9 and 10. In Figure 9, the flexural capacity of shear critical SFRC beam has increased and the neutral axis of the NC and SFRC beams are lower than mid height, i.e., slightly shifted to flexural rebar. The flexural stress is found also increased in flexure critical SFRC beams (Figure 9a and b). The tensile and compressive strength are increased 228% both for flexure critical SFRC beam and in case of shear critical SFRC beam the values are 231% and 239% respectively. The neutral axis in flexure critical beams lies at the mid height as there is no flexural rebar. The models clearly express the influence of higher tensile capacity due to steel fibers. The failure modes are given in Figure 11. The flexure critical beams failed in flexure (Figure 11a and c) but the shear critical beams failed both in flexure and shear mode (Figure 11b and d).



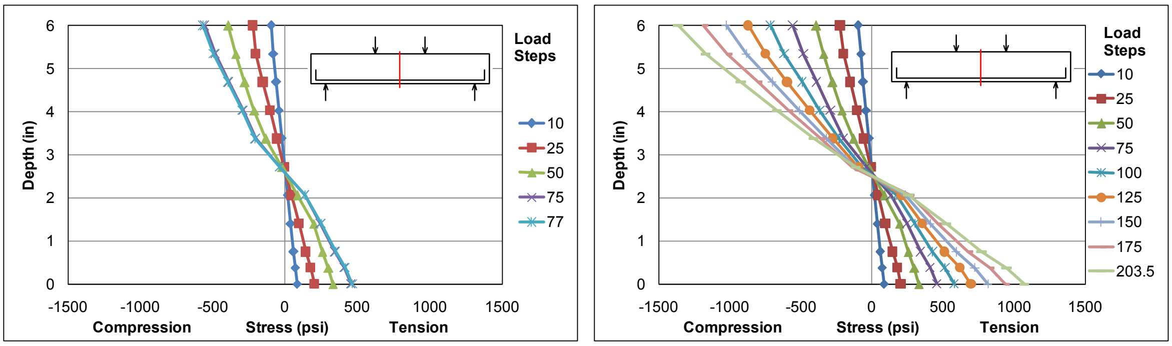
(a) (b)

Figure 7: Shear stress distribution of shear critical beams (a) NC and (b) SFRC



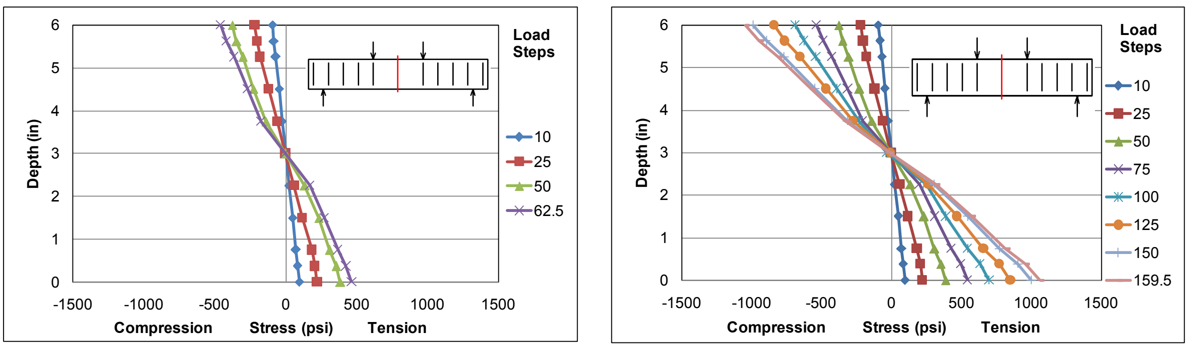
(a) (b)

Figure 8: Shear stress distribution of flexure critical beams (a) NC and (b) SFRC



(a) (b)

Figure 9: Flexural stress distribution of shear critical beams (a) NC and (b) SFRC



(a) (b)

Figure 10: Flexural stress distribution of flexure critical beams (a) NC and (b) SFRC

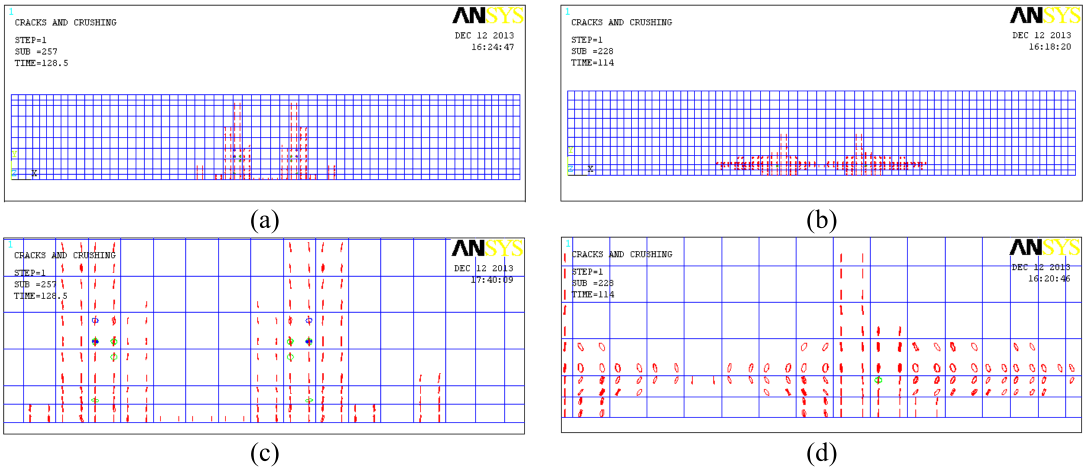


Figure 11: Failure patterns (a) flexure critical NC beam (b) shear critical NC beam (c) close look of flexure critical beam and (d) close look of inclined shear cracks of shear critical beams.

5. CONCLUSION

The following conclusions can be drawn from this investigation:

- The load carrying capacity increased 217% and ductility increased 2.7 times in the shear critical SFRC beam.
- The load carrying capacity increased 200% and ductility increased 2.55 times in the flexure critical SFRC beam.
- The shear stress has increased 204% in flexure critical SFRC beams and 192% in flexure critical SFRC beams.
- The tensile and compressive strength are increased 228% both for flexure critical SFRC beam and in case of shear critical SFRC beam the values are 231% and 239% respectively.
- The flexure critical beams failed in flexure but the shear critical beams failed both in flexure and shear mode.

This paper intended to model the NC and SFRC beams in FE platform and to evaluate the stress distribution due to its increased tensile properties using steel fibers. The models are found capable to analyze the behaviour of the NC and SFRC beams of different reinforcement patterns. These models can be helpful for the construction industry of Bangladesh to use SFRC as engineering material in earthquake resistant structures and to predict their capacity and behaviour.

REFERENCES

ACI 544.4R-88 (1999). “Design Considerations for Steel Fiber Reinforced Concrete”, American Concrete Institute, Reported by ACI Committee 544.

ANSYS Structural Analysis Guide (2005), Version 10.0, ANSYS, Inc., Canonsburg, Pennsylvania.

Aoude, H., Cook, W. D. and Mitchell, D. (2009). “Behavior of Columns Constructed with Fibers and Self-Consolidating Concrete”, ACI Journal, Title no. 106-S34, May-June.

Balaguru, P. and Najm, H. (2004), “High-Performance Fiber-Reinforced Concrete Mixture Proportions with High Fiber Volume Fractions”, ACI Journal, Title no. 101-M31, July-August.

Casanova, P. and Rossi, P. (1997). “Analysis and Design of Steel Fiber-Reinforced Concrete Beams”, ACI Journal, Title no. 94-S54, September-October.

Cheng, M.Y. and Montesinos, G.J.P. (2010). “Evaluation of Steel Fiber Reinforcement for Punching Shear Resistance in Slab-Column Connections, Part II: Lateral Displacement Reversals”, ACI Structural Journal, Title no. 107-S12, January-February.

Dola, J. F., Islam, M. R., Khatun, M. S. and Hussan, M. (2013). “Investigation of the shear capacity of beams made of steel fiber reinforced concrete (SFRC)”, B.Sc. Engineering Thesis, Department of Civil Engineering, Ahsanullah University of Science and Technology, Dhaka, April.

Kachlakev, D.I., Miller, T.Y., Chansawat, S. and Potisuk, T.K. (2001). “Finite Element Modeling of Reinforced Concrete Structures Strengthened With FRP Laminates”, California Polytechnic State University, San Luis Obispo, CA and Oregon State University, Corvallis, OR for Oregon Department of Transportation, May.

Kang, T.H.K., Kim, W., Kwak, Y.K. and Hong, S.G. (2011). “Shear Testing of Steel Fiber-Reinforced Lightweight Concrete Beams without Web Reinforcement”, ACI Journal, Title no. 108-S52, September-October.

Lee, S.C., Cho, J.Y. and Vecchio, F.J. (2011). “Diverse Embedment Model for Steel Fiber-Reinforced Concrete in Tension”, ACI Journal, Title no. 108-M55, September-October.

Traina, L.A. and Mansour, S.A. (1991). “Biaxial Strength and Deformational Behavior of Plain and Steel Fiber Concrete”, ACI Journal, title no.88-M42, July-August.

Uddin, M.K., Mustafiz, A. Z., Chowdhury, M. A. and Mondal, P. (2013). “Investigation of the Flexural Capacity of Beams made of Steel Fiber Reinforced Concrete (SFRC)”, B.Sc. Engineering Thesis, Department of Civil Engineering, Ahsanullah University of Science and Technology, Dhaka, April.

Willam, K.J. and Warnke, E.D. (1975). “Constitutive Model for the Triaxial Behavior of Concrete”, Proceedings, International Association for Bridge and Structural Engineering, ISMES, Bergamo, Italy, Vol. 19, pp 1-30, 1975.

Yakoub, H.E. (2011). “Shear Stress Prediction: Steel Fiber-Reinforced Concrete Beams without Stirrups”, ACI Journal, Title no. 108-S29, May-June.

WEB CRIPPLING STRENGTH OF SINGLE WEB PROFILED STEEL SHEET SUBJECTED TO INTERIOR ONE FLANGE (IOF) LOADING

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ABSTRACT

Profiled steel sheet are used in roofing system as well as affordable quality housing construction due to quickly, efficiently, economically and aesthetically pleasing. However, the webs of profiled steel sheet may cripple due to high localised concentred bearing forces. Web crippling is one of the major failure modes of profiled steel sheet when they are subjected to highly concentrated load or bearing force. In this study, the structural strengths and behaviour of profiled steel sheet subjected to web crippling is investigated. The objective of the research is to determine the web crippling strength and behaviour of profiled steel sheet. The web crippling tests were conducted underIinterior One Flange (IOF) loading condition. The test specimens were fastened to the support. An experimental investigation was conducted on single web members subjected to Interior One Flange (IOF) loading fastened to the bearing plate/support. The test strengths are also compared with the design strengths obtained using North American (AISI S100 2007) Specification. It is shown that the design strengths predicted by these specifications are unconservative for profiled steel sheet subjected to web crippling. The failure loads, failure modes and the load-web deformation behaviour of the profiled steel sheet are presented in this study. It was found that the web crippling capacity of profiled steel sheet may restrict the use of profiled steel sheet. More effective designs can be achieved by taking into account the web crippling strength.

Keywords: Design strengths, Experimental investigation, Interior one flange loading, Profiled steel sheet, Web crippling

1. INTRODUCTION

Profiled steel sheet are increasingly used in structural applications in recent years due to their lightness, corrosion resistance, high strength-to-weight ratio, ease of production, recyclable and availability. Profile steel shell structures are used in roofing elements popularly due to aesthetic and economical use of materials. Roofing consumes a substantial portion of the cost of a building. Thus an economic, efficient and simple roofing system will contribute greatly in solving the problem of housing in general. The roof protects the building and its occupants from the effects of weather, but it is also an architectural feature that gives the building a desired appearance. The roof accounts for a substantial part or about 25% of the total cost of a building (Jagannath and Sekar, 1989). Recent research and development efforts are directed towards developing a light, economical and structurally strong material that can be precast and easily erected (Jagannath and Sekar, 1989). The shape and size of the roofing element is chosen to satisfy the general requirements of strength and stiffness, lightness and economy, ease of handling and erection, proper seating and leak proof joints.Tests on structural strength and behaviour of cylindrical profiled steel sheet proof roofing elements have been performed by Zahurul-Islam et al. (2006). Test results showed that the parabolic profiled steel sheet roofing element provided significant improvements to the roof's structural performance. Howerver profiled steel sheet are often experience web crippling failure due to the high local intensity of concentrated loads or reactions. Web crippling is one of the failure modes that must be taken into consideration in profiled steel sheet design.

In an experimental study at the University of Waterloo, Gerges (1997) investigated on web crippling of single web cold formed C-sections steel members subjected to End-One-Flange Loading. New parameter coefficients for Parabakaran's expression was developed for C-sections subjected to interior one flange loading. Young and Hancock (1998) investigated web-crippling behavior of cold formed steel unlipped channel sections at the University of Sydney. The specimens were tested under four different load conditions of web crippling: End One Flange (EOF), Interior One Flange (IOF), End Two Flange (ETF) and Interior Two Flange (ITF). Based on the test results, the AISI-1996 web-crippling capacity equations were found to be unconservative for the unlipped channel cross sections and a new equation was proposed using a simple plastic mechanism approach.

An experimental study was conducted on web-crippling strength of multiple-web cold-formed steel deck sections subjected to End One Flange (EOF) loading by Onur Avci (2002). Test specimens lying inside and outside of certain geometric parameters of the specifications were tested with both unrestrained and restrained end conditions. The North American Specification (AISI S100 2007) has new web crippling coefficients for different load cases and different end conditions. However, in the End One Flange (EOF) loading case of multi-web deck sections the coefficients for the unfastened configuration were used as a conservative solution for the fastened case. This was because there was no directly applicable test data available in the literature. For that reason, seventy-eight tests were conducted in the Structures and Materials Research Laboratory at Virginia Polytechnic Institute and State University. The web crippling strength of multiple-web cold-formed steel deck sections subjected to End One Flange loading was investigated. The test results were compared with different strength prediction approaches. The study resulted in development of new coefficients for unfastened and fastened multi web deck sections subjected to End One Flange (EOF) loading. Existing studies have revealed that web crippling of thin-walled members could be one of the major failure modes. However, these investigations did not consider single web crippling strength of profiled steel sheet under Interior One Flange.

The main objective of this study is to determine the web crippling strength and behaviour of profiled steel sheet. The structural strengths and behaviour of profiled steel sheet subjected to web crippling is investigated in this study. The web crippling tests were conducted under Interior One Flange loading conditions. The test specimens were fastened to the support. An experimental investigation was conducted on single web members subjected to Interior One Flange (IOF) loading fastened to the bearing plate/support. The test strengths are also compared with the design strengths obtained using both AISI (1996) and North American (AISI S100, 2007) Specification. The design strengths which is predicted by these specifications are unconservative for profiled steel sheet subjected to web crippling. The failure loads, failure modes and the load-web deformation behaviour of the profiled steel sheet are presented in this study. The web crippling capacity of profiled steel sheet may restrict the use of profiled steel sheet on the basis of present results. The web crippling strength may be considered in the design of profiled steel sheet.

2. FIELD APPLICATION OF PROFILED STEEL SHEET FOR ROOFING

The purpose of a structure is to transfer external forces to the supports through structural elements. The trend of modern architecture is now changing from traditional flat surface towards curved shapes that encloses a volume of space. Shells are structures that can be idealized mathematically as curved surface for building roofs. Shells are being used on an increasing scale for roof. Shell element is suitable for roofing because of its efficiency as load carrying member with a high degree of reserved strength and structural integrity, high strength to weight ratio, very small thickness ratio to other dimension, very high stiffness and containment of space. A shell which is formed by translating a curved line along a straight longitudinal axis and which span longitudinally between supporting diaphragms is termed as cylindrical shell. The cylindrical shell is the simplest form of shell, which can be employed in roofing engineering due to its singly curved surface. Figure 1, 2 and 3 illustrates the different applications of profile parabolic shell roof.

Figure 28: Profiled parabolic shell roofing element (Bluedcope-Lysaght, 2003)

Figure 2: Profiled steel sheet roofing element in Sigapore (Bluescope- Lysaght, 2003)

Figure 3: Profiled steel sheet roofing element in Malaysia

3. MATERIAL PROPERTIES

The material properties of the profiled steel sheet were determined by tensile coupon tests. The tensile coupons were extracted from the center of the web plate in the longitudinal direction of the untested specimens. The tensile coupons were prepared and tested according to the American Society for Testing and Materials Standard (ASTM, 1997) and the Australian Standard AS 1391 (AS, 2007) for the tensile testing of metals using 12.5 mm wide coupons. The measured dimensions of the tensile coupons are given in Table 1, according to the nomenclature defined in Figure 4, where b_c is the width of the coupons and t_c is the thickness of the coupons. The coupons were tested in a Universal Testing Machine and the load is applied gradually. Deformation is measured by using deformation gauge. The test set-up and tested specimen after tensile test is shown in Figure 5 and 6 respectively. Figure 7 shows stress–strain curve of tensile coupon.

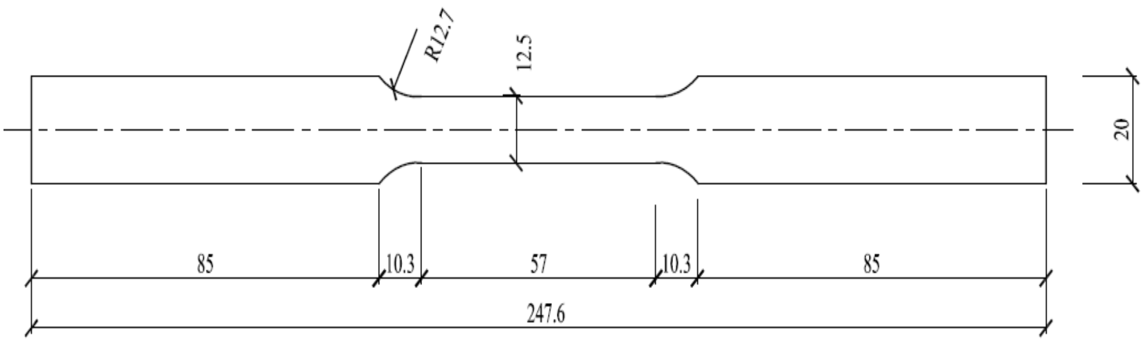

Figure 4: Dimensions of profiled steel sheet tensile coupon test specimen

Figure 5: Profiled steel sheet tensile coupon test

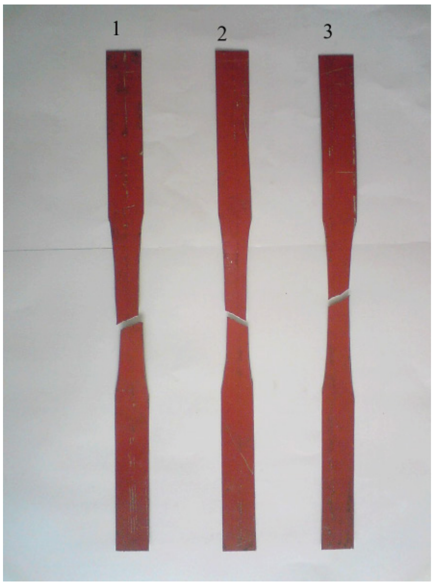

Figure 6: Photograph of test specimen after tensile test

Table 1: Measured material properties obtained from tensile coupon tests

Specimen No.	**Width** b_c **(mm)**	**Thickness** t_c **(mm)**	**Yield Stress** σ_y **(N/mm^2)**	**Ultimate Stress** σ_u **(N/mm^2)**
1.	12.7	0.66	232.7	251.6
2.	12.6	0.65	238.6	261.5
3.	12.7	0.66	244.8	268.2

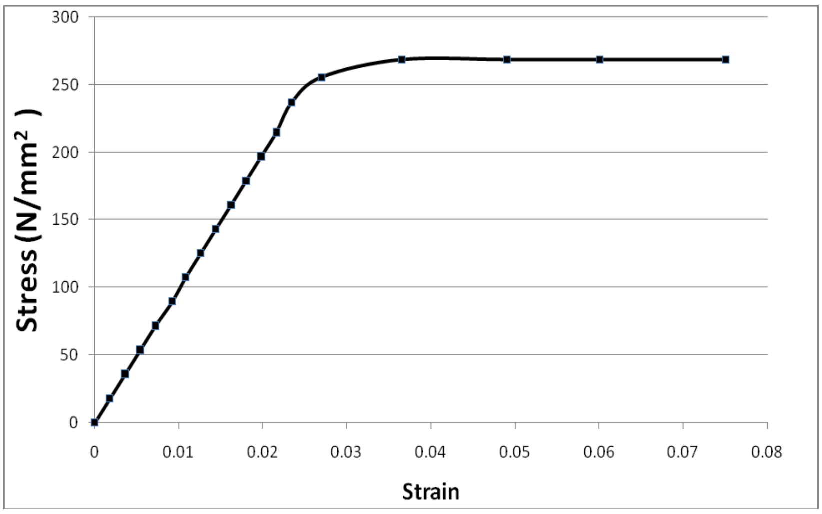

Figure 7: Stress–strain curve of tensile coupon test

4. TEST SPECIMEN

Test specimens lying inside and outside of certain geometric profiled parameter ranges of the specifications were tested under interior one flange loading. The cross section of the steel specimen used in the test is shown in Figure 8. Photograph of test specimen is shown in Figure 9. The measured dimensions of the test specimens subjected to Interior One Flange (IOF) loading are shown in Table 2.

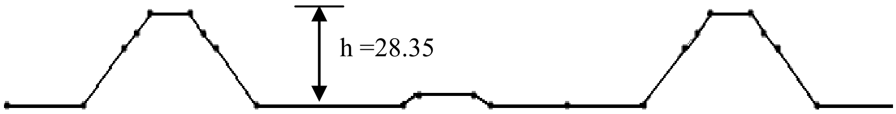

Figure 8: Profiled steel sheet cross section used in the test

Figure 9: Photograph of test specimen before test

In the case of Interior One Flange loading, the length of specimen depends on the length of the bearing plate. According to the Canadian Specification, minimum length of the bearing plate is 1.9 mm. In this test, there are three different length of bearing plate was used, they were 25 mm, 50 mm and 75 mm respectively. Loading digram for IOF load is ahown in Figue 10. Specimen length was used according to North American (AISI S100, 2007) Specification. It was $L= 3h +5N,$ where L = length of the specimen h = heght of the web, and N = baring length. Specimen length according to bearing length is shown in Table 2.

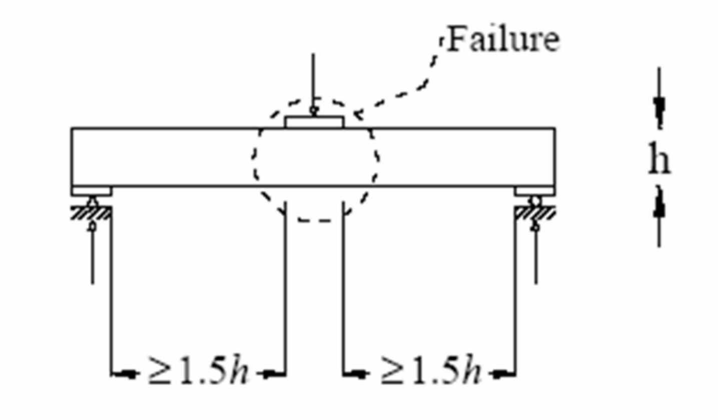

Figure 10: Interior one flange loading diagram

5. WEBCRIPPLING TEST

The web crippling tests were carried out under Interior-One-Flange (IOF) loading condition specified in the ASCE Specification (ASCE 2002). The IOF loading condition of web crippling test is shown in Figure 10. Each specimen is prepared in a similar manner and simulated a simple beam in the entire experiment. The load applied by the ram is simulated as a point load at the mid span location. Photographs of the test setup of IOF loading conditions is shown in Figure 11. A servo-controlled hydraulic universal testing machine was used to apply a concentrated compressive force to the test specimens. In this test, the ends of the sample were bolted to the support. Specimens were fastened to the supports using 11 mm (7/16 in.) bolts with a washer being placed under the bolt head only. The bolt head was always under the bearing plate. All test specimens were supported as shown in Figure 11 and were carefully positioned and aligned in the test frame prior to testing, as shown in Figure 12. The bearing plate was placed under the loading point. The load is applied gradually by using Universal Testing Machine. The speed of the applying load is constant throughout the test for all specimens. Dial gauge was used to recored deformation. The web deformations of the specimens were obtained by the readings of the dial gauge. Load –web deformation was recorded. The maximum load was recorded as the web crippling strength of the specimen under Interior One Flange Loading. The failure modes of specimens is shown in Figure 12 for IOF loading condition. The webs of the specimens were buckled outward.

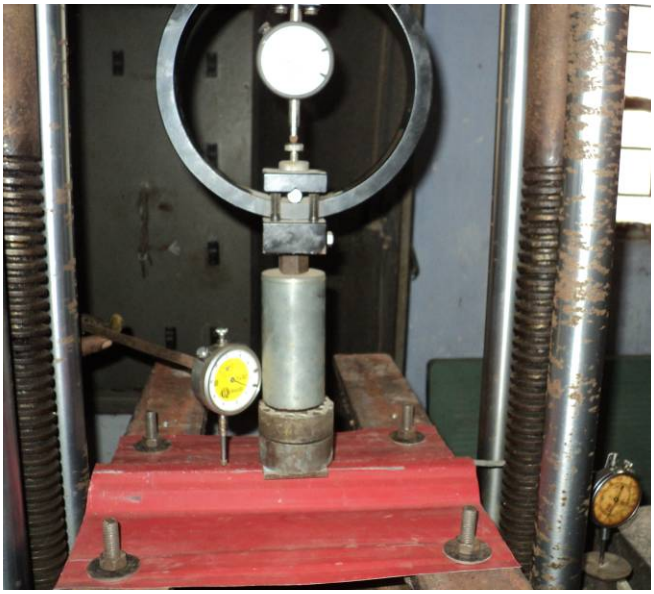

Figure 11: Test setup of web crippling test

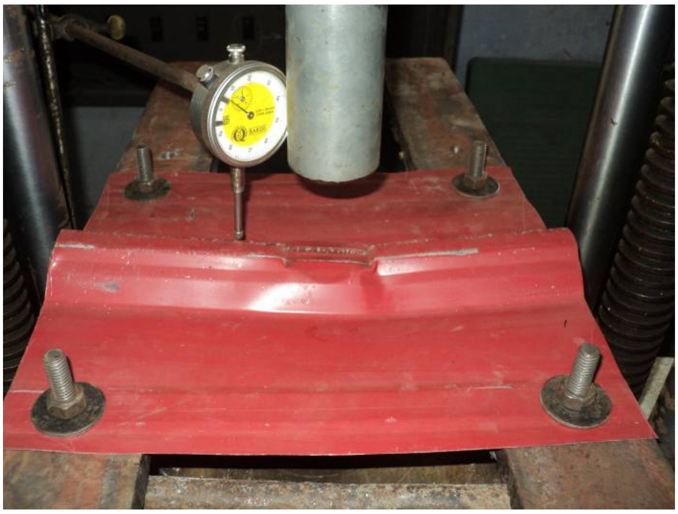

Figure 12: Failure of mode of web crippling test specimen

6. TEST RESULTS AND DISCUSSIONS

The failure modes of profiled steel sheet is shown in Figure 12. It is observed that the specimens tended to fail in the central portions. The progression of crippling on the webs of the specimens initiated at an interior web as the load increased. The crippling of the webs caused deformation on the tension flanges of the specimens and moved the tension flanges upwards. Yu (1981) also observed this type of behaviuor. The amount of resistance provided by the webs was higher in fastened cases than unfastened ones. Load-web deformation behaviour of profile steel sheet using different bearing plate subjected to IOF loading is shown in Figure 13. The maximum load carried by each specimen was recorded as the web crippling strength of the specimen. The web crippling test strengths is shown in Table 2 for IOF loading condition. Observation of the tests revealed that there is an increase in web crippling strength of specimens when bearing length increases. The web crippling strength for IOF loading condition increases as the bearing length increases. The effect of bearing length on the web crippling strength for IOF loading condition is not severe. It is shown that the increases of bearing length do not provide much web cripling strength increases.

Table 2: Web crippling test results

Specimen No.	Length (mm)	Bearing Length (mm)	Thickness (mm)	Web crippling strength *Pt* (kN)
1	210.1	25	0.66	0.891
2	210.3	25	0.66	0.892
3	210.4	25	0.66	0.890
4	335.1	50	0.66	0.958
5	335.0	50	0.66	0.957
6	335.2	50	0.66	0.959
7	460.0	75	0.66	1.031
8	460.2	75	0.66	1.030
9	460.1	75	0.66	1.032

According to the Canadian Specification and using North American coefficient the web crippling strength is determined. The unified North American (AISI S100, 2007) equation is provided in equation 1. The coefficients are given in Table 3.

$$P_n = Ct^2 f_y \sin\theta \left(1 - C_R\sqrt{\frac{r_i}{t}}\right)\left(1 + C_N\sqrt{\frac{N}{t}}\right)\left(1 - C_h\sqrt{\frac{h}{t}}\right) \tag{1}$$

Where C is the coefficient, t is the thickness of the web, f_y is the yield stress ($\sigma_{0.2}$ proof stress), θ is the angle between the plane of the web and the plane of the bearing surface, N is the length of the bearing, h is the depth of the flat portion of the web, C_R is the inside corner radius coefficient, C_N is the bearing length coefficient and C_h is the web slenderness coefficient. The coefficients as well as the respective resistance factor are determined based on the experimental and the numerical results obtained in this study, as shown in Table 3. The limits for the unified web crippling equation for profiled steel sheet web sections when $h/t \leq 200$, $N/t \leq 200$, $N/h \leq 2$ and $\theta = 90°$

Test and analytical attempts have been made to understand web crippling response on profied steel sheet. The profiled steel sheet specimens are tested with Universal testing Machine and maximum crippling strength are obtained. A comparesion is carrid out between the test value (P_t) and the desgn predicted value (P_n) as shown in Table 3. The test value (P_t) and the predicted value (P_n) for the web crippling capacity is not close for most of the fastened specimens. In other words, the ratio, P_t/P_n is not close to 1.0 for most of the fastened data points. Based on the Table 3 results, P_t/P_n is below unity (1). It is shown that the design strengths predicted by these specifications are unconservative for profiled steel sheet subjected to web crippling. Therefore, a unified web crippling equation with existing coefficients for profiled steel sheet under IOF loading conditions shpuld be revised.

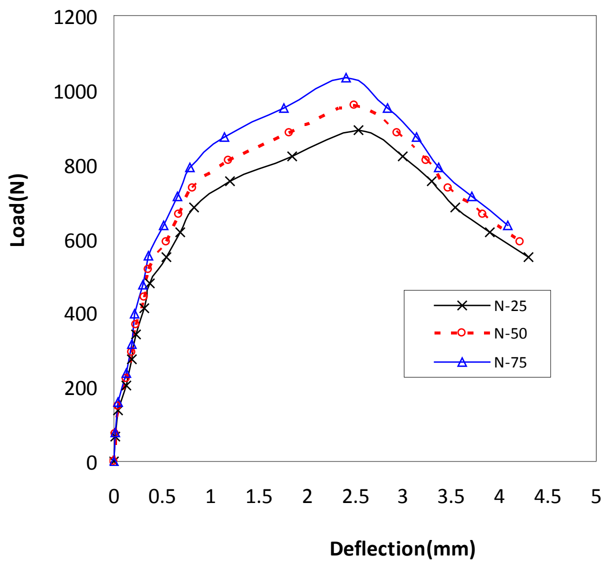

Figure 13: Load-web deformation behaviour of profile steel sheet using different bearing plate subjected to IOF loading

The test results are compared with the predicted design strength values using the *Pt/Pn* ratios for the fastened cases. All test specimens resulted in *Pt/Pn* values lower than unity. North American Specification method resulted in *Pt/Pn* values lower than unity for most of the specimens, meaning that the tested web-crippling values are slightly lower than the predicted web-crippling values. This makes the analytical approaches unconservative. For the North American (AISI S100, 2007) Specification, *Pt/Pn* values which were found to be less than unity belonged to specimens with *R/t* ratios greater than 7.0. Here the test results for all the specimens are lower than analytical results. The web crippling strength of profile steel sheet can be calculated using the North American unified equation with revised the coefficient, and the material properties of longitudinal tension or transverse compression can be used. The web crippling capacity of profiled steel sheet may restrict the use of profiled steel sheet. It is recommendated that the web crippling strength should be considered to acheive more effective designs of profile steel sheet.

Table 29: Web crippling strength calculations with North American Specification (AISI S100, 2007)

Specimen No	t (mm)	f_y (N/mm^2)	h (mm)	R/t	N/t	h/t	N/h	C	C_R	C_N	C_H	P_n (kN)	P_t/P_n
1	0.66	232.7	28.35	19.5	37.9	43.0	0.9	8	0.13	0.10	0.004	0.902	0.99
2	0.66	232.7	28.35	19.5	37.9	43.0	0.9	8	0.13	0.10	0.004	0.902	0.99
3	0.66	232.7	28.35	19.5	37.9	43.0	0.9	8	0.13	0.10	0.004	0.902	0.99
4	0.66	232.7	28.35	19.5	75.8	43.0	1.8	8	0.13	0.10	0.004	1.093	0.88
5	0.66	232.7	28.35	19.5	75.8	43.0	1.8	8	0.13	0.10	0.004	1.093	0.88
6	0.66	232.7	28.35	19.5	75.8	43.0	1.8	8	0.13	0.10	0.004	1.093	0.88
7	0.66	232.7	28.35	19.5	113.6	43.0	2.6	8	0.13	0.10	0.004	1.240	0.83
8	0.66	232.7	28.35	19.5	113.6	43.0	2.6	8	0.13	0.10	0.004	1.240	0.83
9	0.66	232.7	28.35	19.5	113.6	43.0	2.6	8	0.13	0.10	0.004	1.240	0.83

7. CONCLUSIONS

The paper presents an experimental investigation of profiled steel sheet subjected to web crippling. The specimens were tested under Interior-One-Flange (IOF) loading conditions in accordance with the ASCE Specification (2002). The present work deals with web crippling behavior of profiled steel steel sheet. The concentrated load or reaction forces were applied by means of bearing plates. The flanges of the specimens were fastened (restrained) to the bearing plates. The failure modes, failure loads and load-web deformation behaviour of the profiled steel sheet sections have been also presented. It is found that the web crippling strength increases as the bearing length increases subjected to IOF loading. Test and numerical attempts have been made to understand web crippling response of profied steel sheet. The test strengths were compared with the unfactored design strengths obtained using the current North Amerian Specificatiom. The ratio, P_t/P_n value is below 1. It is shown that the numerical design strengths predicted by north Amirican specifications are unconservative. Therefore, the existing coefficients of North Amerian unified web crippling equation for profiled steel sheet under IOF loading conditions can be revised. The design strengths should be calculated using the material properties obtained from coupon tests. The web crippling capacity of profiled steel sheet may restrict the use of profiled steel sheet in field application. It is recommendated that the web crippling strength should be considered to acheive more effective designs of profile steel sheet.

ACKNOWLEDGEMENTS

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REFERENCE

AISI S100 (2007). North American Specification for the design of cold-formed steel structural members. North American Cold-formed Steel Specification, American Iron and Steel Institute, AISI S100-2007, Washington, D.C., 2007.

AISI (1996). Specification for the design of cold-formed steel structural members, American Iron and Steel Institute, Washington, D.C. 1996.

AS. Methods for tensile testing of metals, Australian standard AS 1391. Sydney, Australia: Standards Association of Australia, 1991.

ASCE. Specification for the design of cold-formed stainless steel structural members.SEI/ASCE 8-02; Reston, VA: American Society of Civil Engineers, 2002.

ASTM. Standard test methods for tension testing of metallic materials, E 8M-97. West Conshohocken: American Society for Testing and Materials, 1997.

Bluescope- Lysaghat Malaysia Sdn Bhd. (2003) “Information and Brochures”, Kuala Lampur, 2003.

Gerges, R. R. (1997). “Web Crippling of Single Web Cold Formed Steel Members Subjected to End One-Flange Loading”, M.Sc. Thesis, University of Waterloo, Waterloo, Ontario, Canada.

Jagannath, V. and Sekar, U. C. (1989). “Precast L-pan Roof Elements for Low Cost Housing.” Journal of Ferrocement Vol. 19(4), 353-359.

Onur Avci (April, 2002) “Web- Crippling Strength of Multi-Web Cold-Formed Steel Deck Sections Subjected To End One Flange (EOF) Loading Web.” Thesis Submitted to the faculty of the Virginia Polytechnic Institute and State University

Young, B. and Hancock, G .J. (1998). “Web Crippling Behavior of Cold Formed Unlipped Channels”, Fourteenth International Specialty Conference on Cold Formed Steel Structures, University of Missouri-Rolla, Rolla, Missouri, 127-150.

Yu, W.W. (2000). ”Cold Formed Steel Design”, Third Edition, John Wiley & Sons, Inc., New York, NY.

Zahurul-Islam S. M., Abang-Abdullah A.A and Jafar M. S. (2006), “An Investigation on Structural Performance of Profiled Steel Sheet to Develop Self-supporting Roofing System.” The International Journal of Advanced Steel Construction,Vol. 2 (1): 87-108.

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A COMPARATIVE STUDY ON SEISMIC ANALYSIS BY BANGLADESH NATIONAL BUILDING CODE (BNBC) WITH OTHER BUILDING CODES

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ABSTRACT

Tectonic framework of Bangladesh and adjoining areas indicate that Bangladesh lies well within an active seismic zone. The after effect of earthquake is more severe in an underdeveloped and a densely populated country like ours than any other developed countries. Bangladesh National Building Code (BNBC) was first established in 1993 to provide guidelines for design and construction of new structure subject to earthquake ground motions in order to minimize the risk to life for all structures. A revision of BNBC 1993 is undergoing to make this up to date with other international building codes. This paper aims at the comparison of various provisions of seismic analysis as given in building codes of different countries. This comparison will give an idea regarding where our country stands when it comes to safety against earth quake. Primarily, various seismic parameters in BNBC 2010 (draft) have been studied and compared with that of BNBC 1993. Later, both 1993 and 2010 edition of BNBC codes have been compared graphically with building codes of other countries such as National Building Code of India 2005 (NBC-India 2005), American Society of Civil Engineering 7-05 (ASCE 7-05). The Base Shear/Weight ratios have been plotted against the height of the building. The investigation in this paper reveals that BNBC 1993 has the least base shear among all the codes. Factored Base shear values of BNBC 2010 are found to have increased in the range of 6-68 percent than that of BNBC 1993 for low rise buildings (≤ 20 m) around the districts than its predecessor. Despite revision of the code, BNBC 2010(draft) still suggests less base shear values when compared to the Indian and American code. Therefore, this increase in factor of safety against the earthquake imposed by the proposed BNBC 2010 code by suggesting higher values of base shear is appreciable.

Keywords: BNBC codes, NBC-India 2005, ASCE 7 05, Base Shear, Height

1. INTRODUCTION

Bangladesh lies well within an active seismic zone and is prone to earthquakes. To determine earthquake forces on a structure, static analysis has gained popularity in the country and also in many other countries because of the simplicity of the method. This calls for the use of an established and tested building code so as to ensure the safety of the structure and its occupants against the natural hazard. Bangladesh National Building Code (BNBC) was first organized in the year of 1993 to fulfill the purpose. As the number of high rise buildings is increasing, the international codes followed for building design, detailing and construction is revised quite frequently to adopt the new practices. Initiative has already been taken to update BNBC 1993 and a draft copy has already been prepared (BRTC & BUET, 2010). A total change at wind load and earthquake provisions in the proposed code can be noticed. This paper is aimed to review and compare the current and proposed seismic design provisions dealing with the specification of seismic design forces among the existing and recently proposed BNBC codes as well as other codes of different countries.

2. OBJECTIVE

The main objective of this thesis is to compare the current seismic provision with the coming one as well as well known seismic provisions of other countries. In detail, the objectives are:

1. To become familiar with new seismic design methodology as described in BNBC 2010 (Draft)
2. To compare similarities as well as differences between BNBC 1993 and BNBC 2010 (Draft)
3. To compare BNBC code (both the existing and proposed editions) with National Building Code of India 2005 (NBC-India 2005) and American Society of Civil Engineering (ASCE 7 05)

3. METHODOLOGY

The method of calculation of seismic loading is more or less same in BNBC 2010 (draft) and BNBC 1993. Both these codes consider the earthquake force as a lateral force. The forces are determined on the basis of a base shear by Equivalent Lateral Force procedure. Base shear is calculated on the basis of seismic zone factor, structural importance factor and response reduction factor which is a function of structural system. Time period and soil type as a function of acceleration spectrum (C_s) defined by BNBC 2010 and as a function of numerical coefficient (C) defined by BNBC 1993 are used in the expression of base shear. The base shear/weight ratios have been compared graphically with respect to the height of the building. NBC-India 2005 code also follows somewhat similar approach. Base shear/Weight ratios are computed from the given formula and hence plotted against height.

Like all modern codes, zone factor is replaced in ASCE 7 05 code by two numbers S_s and S_1 for the specific geographic location (latitude and longitude) of the project site representing response spectrum acceleration at shorter and 1s period respectively as a percentage of gravity. Electronic values of mapped acceleration parameters all around the world are found at the USGS web site at http://earthquake.usgs.gov/designmaps and are directly used in the base shear expression to plot a comparative graph with the BNBC codes.

Table1: Seismic parameters in a nutshell

	Equivalent Terms in Four Different Standards			
	BNBC 1993	**BNBC 2010**	**NBC-India 2005**	**ASCE 7 05**
Base Shear Formula	$V = \frac{ZIC}{R}W$	$V = \frac{2}{3}\frac{ZIC_s}{R}W$ $\geq \frac{2}{3}\beta Z$ where, β=0.2	$V_B = \frac{ZI}{2R} \times \frac{S_a}{g}W$	$V = \frac{S_{DS}}{R/I_e}W$ $\leq \frac{S_{D1}}{T(R/I_e)}W$
Zone Factor	Z=Seismic Zone Coefficient	Z=Seismic Zone Factor	Z=Seismic Zone Factor	
Importance Factor	I=Structure Importance Coefficient	I=Structure Importance Factor	I= Importance Factor	I_e= Importance Factor
Structural System Factor	R=Response Modification Coefficient	R=Response Reduction Factor	R=Response Reduction Factor	R=Response Modification Factor
Structural Response Factor	$C = \frac{1.25S}{T^{2/3}}$ C=Numerical Coefficient, T=Time Period, S=Site Coefficient	C_s=Normalized Acceleration Response Spectrum=f(time period T, Soil Factor S and Damping Factor)	S_a/g=Average Response Acceleration Coefficient= f(time period, T)	T=Time Period
Effective weight of Structure	W=Total Dead Load + some specified live loads	W=Total Dead Load + some specified live loads	W=Total Dead Load + some specified live loads	W=Total Dead Load + some specified live loads
				S_{DS} and S_{D1}= Design Spectral Response Acceleration Parameter at shorter period and at 1s respectively

4. COMPARISON OF VARIOUS PARAMETERS

4.1 Seismic Zone Factor (Z)

On the basis of distribution of earthquake epicentres, ground motion attenuation, geophysical and tectonic data available from within as well as outside of the country, Bangladesh was mapped dividing into three generalized seismic zones in BNBC 1993 (Ali, 1998). The seismic zoning map is revised in the proposed BNBC 2010 with provisions for four seismic zones with different level of ground motion. Each zone has a seismic zone coefficient (Z) which represents the maximum considered peak ground acceleration (PGA) on very stiff soil/rock (site class SA) in units of g (acceleration due to gravity The north-eastern folded regions of Bangladesh are the most active zones and has a maximum PGA value of 0.36g. Therefore, northern east areas of the country are given highest priority in the both existing and proposed edition of BNBC. Seismic zone factor is increased considerably in BNBC 2010.

4.2 Structural Importance Factor (I)

The Importance Factor (I) provides higher seismic force level for facilities deemed essential to public welfare and should remain functional for use after a major earthquake. However, the role of Importance Factor may be made less significant if design details could be ensured in the code of practice. In BNBC 1993, structure importance co-efficient is separated for structural and non-structural components and equipment and denoted by I and I´ respectively. But in BNBC 2010, importance co-efficient is denoted by I for all cases. The descriptions of the cases in old and new codes are completely different. Importance co-efficient is found higher in BNBC 2010 and increased up to 25% for some cases.

4.3 Soil Factor (S)

The zone factors are the representation of ground acceleration on rock soil. To take into account other soil conditions, soil factor (S) is introduced. The amount of ground motion amplification depends on wave-propagation characteristics of soils, which can be estimated from the measurements of shear wave velocity. Soft soils with slower shear wave velocities generally produce greater amplification than stiff soils with faster shear wave velocities (Nikolaou. S, 2008).

The site classes are defined mainly in terms of soil profile depth and shear wave velocity in the existing code. BNBC 2010 includes additional two procedures to determine the site classes as measuring shear wave velocity adds cost to a geotechnical investigation. Such classification is based on Standard Penetration Resistance, Undrained Shear Strength. In both codes, the effect of site geology and soil characteristics is incorporated in the expression of numerical coefficient (C) or acceleration response spectrum (Cs).

4.4 Response Reduction Factor (R)

It would be economically unfeasible to design a building to remain elastic in all level of earthquake. To account for inelastic behaviour, response reduction or modification factor is introduced. It is the factor by which the actual base shear force that would develop if the structure behaved truly elastic during earth quake is reduced to obtain design base shear. This reduction is allowed to account for the beneficial effects of inelastic deformation that can occur in a structure during a major earth quake. The value of response modification factor is reduced on the order of 33 percent in BNBC 2010 for different structural systems (Table 4.3). This reduction is logical since only two-third of the maximum considered earthquake (MCE) ground motion is considered to be design basis earth quake (DBE) in BNBC 2010 rather than full MCE in BNBC 1993.

Table 30: Comparison of Response Reduction Factor between BNBC 1993 and 2010

Structural System	Description of Lateral Force Resisting System	BNBC 1993	BNBC 2010	Percent Decreased
Building Frame System	1. Steel eccentrically Braced Frame	10	8	20
	2. Reinforced Concrete Shear wall	8	5	37.5
	3. Special Reinforced Concrete Shear wall	-	6	-
	4. Ordinary Steel Concentrically Braced frame	-	3.25	-
	5. Special Concentrically Braced frame	8	6	25
Moment Resisting Frame System	1. Special Steel Moment Resisting Frame	12	8	33.33
	2. Special Concrete Moment Resisting Frame	12	8	33.33

	3. Intermediate concrete Moment Resisting Frame	8	5	37.5
	4. Ordinary Steel Moment Resisting Frame	6	3.5	41.67
	5. Ordinary Concrete Moment Resisting Frame	5	3	40

4.5 Time Period (T)

The fundamental building period is simply the inverse of the building frequency at which it wants to vibrate when set in motion by some sort of disturbance (in building design, typically a seismic or wind event) based on the system's mass and stiffness characteristics. Buildings with shorter fundamental periods attract higher seismic forces as the calculation of overall base shear diminishes with increasing building period.

Precise determination of time period, as required in base shear calculation, often constitutes certain difficulties as it is time consuming as well as complex. There is very little need to determine the time period accurately since response spectra is insensitive to the small changes in the time period of structure (Noor, Ansary and Siraj, 1997). Therefore, almost all codes specify empirical formulas for the determination of period of structure.

The building period T may be approximated in both the codes by the following formula:

$$T = C_t h_n^m ,$$ h_n = height of the building in m.

The value of C_t and m are different in the two codes.

Table 3: Comparison of Time Period

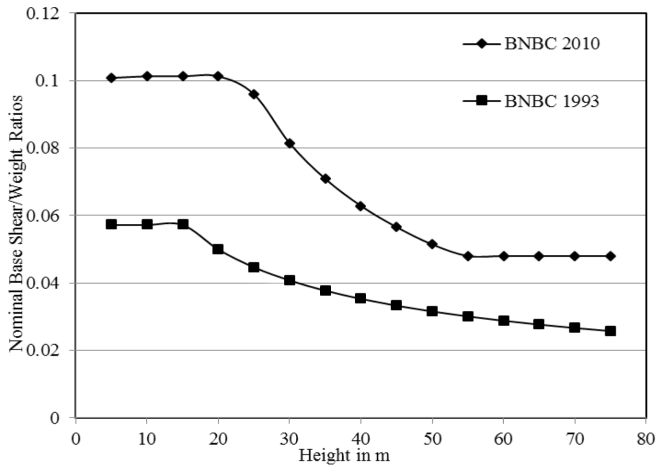
Structure Type	**BNBC 2010**		**BNBC 1993**	
	C_t	**m**	**C_t**	**m**
Concrete Moment Resisting Frame	0.0466	0.9	0.073	0.75
Steel Moment Resisting Frame	0.0724	0.8	0.083	0.75
Eccentric Based Steel Frame	0.0731	0.75	0.073	0.75
All Other Structural Systems	0.0488	0.75	0.049	0.75

4.6 Normalized Response Spectrum Acceleration (C or C_s)

It is the response of a building due to ground acceleration. Code-based response spectrum is similar to Numerical coefficient of BNBC 1993 in a sense that both are functions of time period (T) and soil type (S). But 2010 edition of the code introduces an additional parameter Damping Factor as a function of response spectrum. Damping factor is the effect of inherent energy dissipation mechanisms in a structure (due to sliding, friction, etc.) that results in reduction of effect of vibration, expressed as a percentage of the critical damping for the structure. BNBC 2010 suggests that 5% damped design spectrum to be properly modified for an actual damping factor. BNBC 2010 introduces four equations each operating within a range of time period to determine Cs.

5. BASE SHEAR AND ITS DISTRIBUTION

A comparison of base shear is the simplest way to compare the final result. But to make it more generalized, the base shear/weight ratios are plotted against the height of the building. Only RC ductile moment resisting framed buildings have been dealt with, because of the wide use of this structure in Bangladesh. The zones with similar seismic activity are considered and loose soil condition has been assumed. For simplicity, building with no horizontal and vertical irregularities has been considered. Sylhet being the region of the highest seismicity in Bangladesh is considered as the location for the mapped zone factor in figure 1.



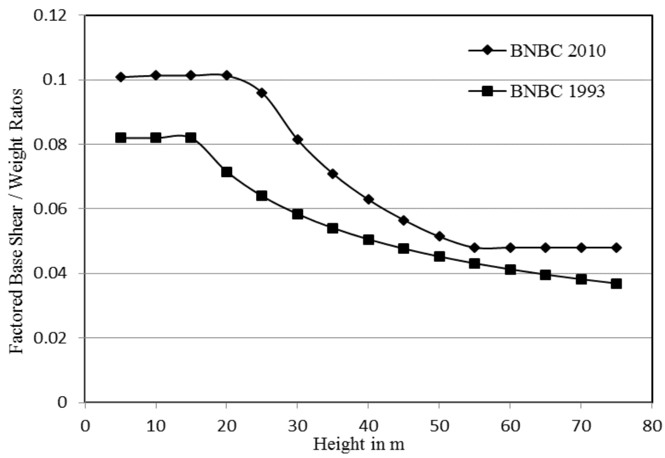
BNBC 2010	BNBC 1993
I=1 Z=0.36 SD=1.35 R=8	I=1 Z=0.25 S3=1.5 R=12

Figure 31: Seismic base shear comparison for RC Special Moment Resisting Frames (SMRF) for loose soil condition in Sylhet

In Ultimate Strength Design, the nominal earthquake loads are multiplied by a factor called Load Factor. They remain in combination with other loads and termed as Factored Load. The earthquake load combinations of the codes compared here are shown in the table 4.5. After incorporating the corresponding maximum load factors of the BNBC 1993 and BNBC 2010, the previous graph is reconstructed and found like figure 3.

Table 4: Comparison of seismic load cominations in USD method

BNBC 1993	**BNBC 2010**	**NBC-India 2005**	**ASCE 7 05**
1.05D+1.275L+1.4E	1.2D + 1.0E + 1.0L	1.2D+1.2L+1.2E	1.2D + 1.0E + 1.0L
0.9D+1.43E	0.9D + 1.0E	0.9D+1.5E	0.9D + 1.0E



BNBC 2010	BNBC 1993
I=1 Z=0.36 SD=1.35 R=8	I=1 Z=0.25 S3=1.5 R=12

Figure 32: Seismic factored base shear comparison for RC Special Moment Resisting Frames (SMRF) for loose soil condition in Sylhet

There has been in the range of 6-68 percent increase in factored base shear of BNBC 2010 at low rise buildings (≤ 20 m) around various districts. At higher altitude, however, the difference between the two codes becomes very small. The above plots will alter amplitude for the different locations, site classes and structural systems of the building in question but the nature of the curve remains the same. Therefore, BNBC 2010 surely imposes higher base shear than BNBC 1993 for buildings irrespective of site classes and structural systems.

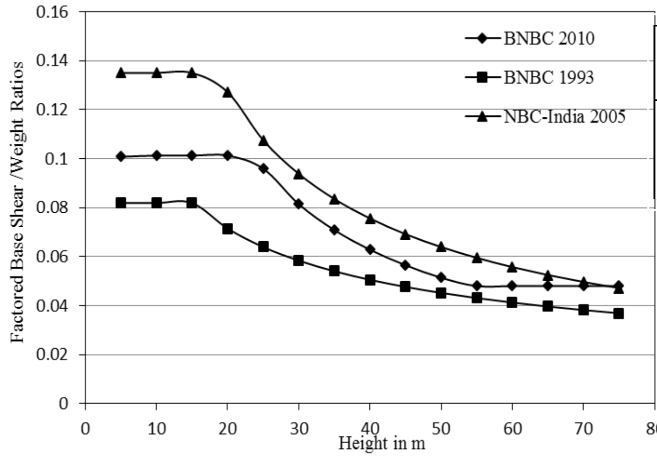
The vertical distribution of seismic forces of BNBC 2010 is different from that of BNBC 1993. The BNBC 2010 prescribes a linear distribution and a parabolic distribution for structures with $T < 0.5$ seconds and $T > 2.5$

seconds respectively varying from a zero value at the base to a maximum value at the top. For intermediate periods, one may use a linear interpolation between a linear and a parabolic distribution, or a parabolic distribution which is more conservative. The BNBC 1993 uses a linear distribution, with zero value at the base, for structures with $T < 0.7$ seconds. For longer-period structures, a portion of the design base shear ($0.07TV \leq 0.25V$) is concentrated at the top, with the remainder of the design base shear being distributed linearly as for short-period structures.

6. COMPARISON OF BNBC WITH OTHER BUILDING CODES

6.1 With NBC-India 2005

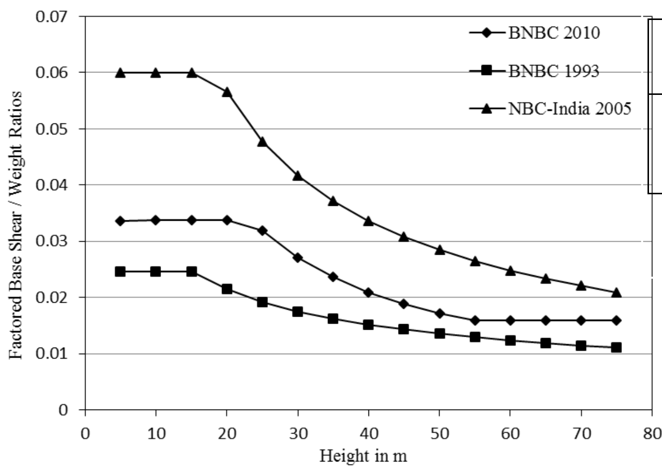
Northern part of Bangladesh is surrounded by the regions of high seismicity which includes the Shillong plateau having possessed by multiple faults. Since these parts of Bangladesh and India can be characterized by the same tectonic features, computation of base shear for those zones following respective codes will highlight the design standards of these two countries against earthquake. Both BNBC (1993 and 2010) and NBC- India 2005 put the highest priority in these most severe earthquake prone zones by suggesting the highest seismic zone factor, Z= 0.36. Consideration of Soil with identical geotechnical features is important in comparing the base shear values as different types of soil subject to different sorts of ground motion. Soil type S3, SD and soft soil condition as per BNBC 1993, BNBC 2010 and NBC-India 2005 respectively is assumed for their identical geotechnical characteristics (N<15, where N=Standard Penetration Value). The maximum load factors of 1.43,1 and 1.5 against earthquake for BNBC 1993, BNBC 2010 and NBC-India 2005 are taken into account to plot the graphs.



BNBC 2010	BNBC 1993	NBC India 2005
I=1 Z=0.36 SD=1.35 R=8	I=1 Z=0.25 S3=1.5 R=12	I=1 Z=0.36 Soft Soil R=5

Figure 3: A comparison of factored base shear between BNBC and NBC-India 2005 codes for Sylhet and Shillong regions at loose soil condition

The above graph is a plot of factored base shear for the maximum seismic loading that is governed by each of the respective codes and it shows proposed BNBC code exceeds the Indian code by some margins. An-other graph is plotted below showing the seismic base shear for Jessore and Kolkata having similar tectonic and geological features but defined as low and moderate seismic intensity zone respectively in the respective codes. Due to the absence of IMRF in the building system of the Indian code, SMRF as a lateral load resisting system is considered for moderate seismic risk. Soil type is assumed as before. Kolkata is found higher in terms of factored base shear.



BNBC 2010	BNBC 1993	NBC India 2005
I=1 Z=0.36 SD=1.35 R=8	I=1 Z=0.25 S3=1.5 R=12	I=1 Z=0.36 Soft Soil R=5

Figure 4: A comparison of factored base shear between BNBC and NBC-India 2005 codes for Jessore and Kolkata cities at loose soil condition

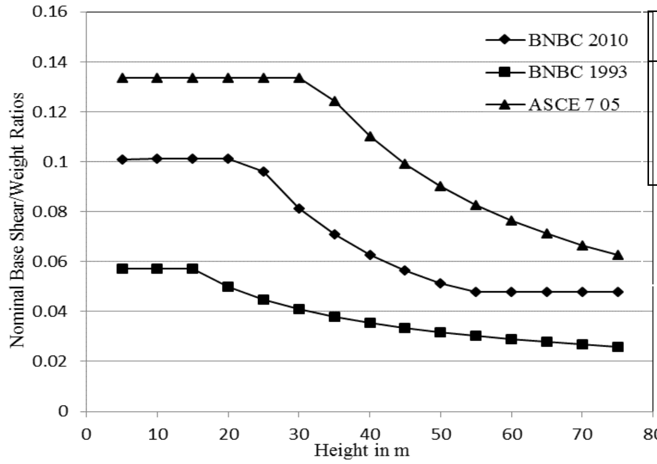
Fundamental difference in the Indian and Bangladesh standard may be attributed to the fact that the design earth quake of BNBC 2010 is two-third of Maximum Considered Earthquake (MCE) while the Indian standard designs with one half of MCE. This indicates buildings in Bangladesh will collapse form an earthquake that is 1.5 times larger than design earthquake while buildings in India will collapse from 2 times larger earthquake than corresponding design earthquake.

6.2 With ASCE 7 05

The ASCE 7 Code is the basis of most state seismic codes. The seismic criteria of International Building Code (IBC) are taken from ASCE 7. ASCE 7 Code provides seismic ground motion parameters as spectral acceleration coefficients S_s and S_1 (spectral response accelerations at 0.2 and 1.0 seconds, respectively, for 5% of critical damping) assuming Site Class "B" with 2% probability of exceedance in 50 years. For the ASCE 7 05, these acceleration values come from the 2002 United States Geological Survey (USGS) seismic hazard maps, which are available at http://earthquake.usgs.gov/designmaps.

Worldwide Seismic Design maps Web Application provides earthquake shaking parameters, more specifically, Ss and S1 values worldwide that are needed for seismic design of structures using the ASCE 7 code and similar standards (e.g., the IBC/SEI standard). This application mainly uses gridded data from the Global Seismic Hazard Assessment Program (GSHAP) project for South Asia region. The GSHAP data cover much of the world with the major exceptions being vast expanses of ocean. Ss and S1 values are approximate values based on the probabilistic 10%-in-50-year peak ground accelerations (PGA's) from GSHAP. The GSHAP values are multiplied by 2 to approximate 2%-in-50-year PGA values, and then multiplied by 2.5 and 1.0, respectively, to estimate Ss and S1. This application is used here as a tool to gather a preliminary assessment of the seismic design parameters for Bangladesh to make a comparison with the ASCE 7 05 standard.

According to the ASCE 7 05, each building is assigned to one of the six structural design categories (SDC) depending on risk category and the values of Ss and S1. On such basis, Sylhet is found to be under SDC D. The base Shear/weight ratios are plotted for Sylhet following Equivalent Lateral Procedure which is applicable to a SDC D category building with no certain vertical or horizontal irregularities unless T>3.5Ts (Table 12.6-1,ASCE 7 05). ASCE 7 05 is found to be highly conservative. Since BNBC 2010 and ASCE 7 05 has similar load factor, graph is plotted on the basis of nominal base shear.



BNBC 2010	BNBC 1993	ASCE 7 05
I=1 Z=0.36 SD=1.35 R=8	I=1 Z=0.25 S3=1.5 R=12	I=1 Site SE S_s=1.78 S_1=0.71 R=8

Figure 5: A comparison of base shear between BNBC (both 1993 and 2010) and ASCE 7 05 codes at loose soil condition for Sylhet

7. CONCLUSION

BNBC 1993 suggests the least base shear values among the current codes compared in this paper. While developed countries are going for more conservative design, this contradiction of BNBC 1993 could be suicidal. Some modifications are needed to be made in this respect. Proposed BNBC 2010 will surely be a more conservative approach in the seismic design of buildings in Bangladesh. Base shear values of BNBC 2010 are found to have increased in the range of 6-68 percent than that of BNBC 1993 for low rise buildings (≤ 20 m) around the districts.

But BNBC 2010 has 32-36 percent less base shear value as compared to ASCE 7 05 for low storied buildings (≤ 20m). Because the ASCE 7 code design parameters are generic, they also generally impose higher base shear values. As India is the closest neighbour to Bangladesh and shares the same tectonic zone, comparison with the Indian standard will be of more significance. Looking at the Indian standard, the design seismic loading set by BNBC 2010 seems to be well justified as the nominal base shears in the proposed standard are relatively closer to that of NBC-India 2005.

Therefore, this increase in factor of safety against the earthquake imposed by the proposed BNBC 2010 code by suggesting higher values of base shear is appreciable. But remarkably higher reinforcement requirement in ground floor column of low storied buildings than before might be a concern for building design in Bangladesh by the proposed code. Further studies need to be made in this aspect.

ACKNOWLEDGEMENTS

First of all, I would like to thank my family members for their unconditional support. I am indebted to and wish to express my utmost gratitude to my supervisor Professor Dr. Md. Shafiul Bari for his continuous assistance and invaluable insight in supervising this work. Last but not the least, a special thank is attributed to my friend Rayhan Md. Faysal for helping me out with all the necessary information and precious suggestions.

REFERENCES

Atique, F. and Wadud, Z. (2001). “A comparison of BNBC-93 with other building codes with respect to earthquake and wind analysis.” The Eighth East Asia-Pacific Conference on Structural Engineering and Construction, Nanyang Technological University, Singapore.

American Society of Civil Engineering 7 05 code, ASCE 7 05. “Chapter 11 Seismic Design Criteria and Chapter 12 Seismic Design Requirement of Building Structures” published by American Society of Civil Engineers 1801 Alexander Bell Drive Reston, Virginia 20191

BRTC and BUET, 2010.”Updating of Bangladesh National Building Code.” Inspection Report submittted to HBRI. Retrived from http://www.buet.ac.bd/ce/pdf/Top%20pages_Inception%20Report.pdf

Bangladesh National Building Code, BNBC 1993. “Part 6 Chapter 2 Section 2.5, Earth Quake Loads” published by Housing and Building Research Institute and Bangladesh Standard and Testing Institute, Bangladesh.
Bangladesh National Building Code, BNBC 2010 (Draft). “Part 6 Chapter 2 Section 2.5, Earth Quake Loads.”
Information on http://earthquake.usgs.gov/hazards/designmaps/wwdesigndoc.php
Kelly, D. (2006) “Seismic site classification for structural Engineers”, http://www.structuremag.org/Archives/2006-12/SF-SEISMIC-Dec06-p21-24.pdf
National Building Code-India 2005, “Part 6 Section1, Load, Forces and Effects” published by Bureau of Indian Standards, New Delhi.
Nikolaou, S. (2008). “Site Specific Seismic Study for Optimal Structural Design”, http://www.structuremag.org/Archives/2008-2/C-StructuralDesign-Nikolaou-Feb08.pdf
Noor, M.A. ; Ansary, M.A. and Siraj, S.M. (1997) “Critical Evaluation and Comparison of Different Seismic Code Provisions.” Journal of Civil Engineering, The Institute of Engineers, Bangladesh Vol. CE 25, No. 1
Worldwide Seismic Design Maps Web Application available at https://geohazards.usgs.gov/secure/designmaps/ww/signup.php

PERFORMANCE OF STONE DUST AS PARTIALLY REPLACING MATERIAL OF BINDING MATERIAL AND FINE AGGREGATE ON STRENGTH PROPERTIES OF MORTAR

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ABSTRACT

Morter is an integral part of reinforced concrete structural member. Due to Rapid growth in construction activity, the consumption of mortar is increasing every year. This results in excessive use of natural aggregates and binding materials. This is why it is becoming inevitable to use alternative materials for aggregates in mortar. This research is conducted to study the effect of stone dust as replacing material of both fine aggregate and binding material subjected to different water-binder ratio on the strength properties of mortar. An endeavor has been made to determine the applicability of stone dust as construction material. In this work two types of fine aggregate were used and replaced partially (0%, 25%, 50%) by stone dust. At the same time the cement was also replaced partially (0%, 5%, 10%) by stone powder. Two types of sand were used in the study and the coarser one (FM 2.74) exhibit higher strength value than the finer one (FM 1.22). In case of replacement of fine aggregate up to 25% exhibits highest strength than the controlled sample. The strength of mortar gradually decreases with the replacement of cement by stone powder.

Keywords: Strength, Stone Dust, Stone powder, Binding material, Fine Aggregate

1. INTRODUCTION

Mortar is a widely used construction material consisting of cementing material, fine aggregate and required quantity of water, where in the fine aggregate is usually natural sand. Mortar should be distinguished from concrete, which acts in a similar way of mortar but which contains coarse aggregate which is bound together by the cement (T. Hoque et al.,2013). Mortar provides distribution stresses and protection to the outer part of structural concrete from detrimental effect. Some Portland cement assists the workability and plasticity of the mortar. It also provides early strength to the mortar and speeds setting. Sand is the general component of mortar which gives its distinctive color, texture and cohesiveness. The strength of mortar or concrete is largely affected by the fine aggregates (Sharmin et al., 2006). Fine aggregate is usually sand from river (Lohani et al., 2012). The main constituents of mortar is sand are mainly natural resources (H.M.A.Mahzuz,et al.,2011). Available sources of natural sand are getting bushed due to swift growth of construction activity (Palaniraj, 2003). The use of sand in construction results in excessive sand mining which is objectionable. Due to rapid growth in construction activity, the available sources of natural sand are getting exhausted. Alternative material of sand should be explored to mitigate the increasing demand of sand. A considerable amount of dust is produced at the time of stone crushing. They are often considered as a waste in the locality. Saving of natural resources and environment is the essence of any advancement (M.Veera Reddy, 2010). Exchange of normal sand by stone dust will assist both solid waste minimization and waste recovery (H.M.A.Mahzuz, 2011). Replacement of fine aggregate in mortar by crushed rock powder was studied giving a result that the strength of modified mortar is 40% higher than the normal mortar (Nagabhushana and H.Sharada bai, 2011). An investigation was performed by M.Haque (2012) on the strength properties of mortar using powder material which is generated from stone. An experimental investigation was done by Er. Lakhan Nagpal et al, (2013) titled “Evaluation of Strength Characteristics of Concrete Using Crushed Stone Dust as Fine Aggregate”. This study reported that the strength of Quarry Rock Dust concrete is comparatively 10-12 percent more than that of similar mix of Conventional Concrete. It is found that natural river sand, if replaced by hundred percent quarry rock dust from quarries, may sometimes give equal or better than the reference concrete made with Natural Sand, in terms of compressive strength (R. Ilangovana et al, 2008). Analysis of experimental data showed that the addition of the quarry dust improved the strength properties of concrete (Sivakumar and Prakash M. 2011). Athours reported that replacement of fine aggregate by crusher dust up to 50% by weight has a negligible effect on the reduction of compressive strength (Radhikesh P. Nanda et al, 2010).

Now-days, all over the world, construction activities are taking place on huge scale. Due to this there is great increase in cost of construction. Natural river sand is one of the key ingredients of concrete, is becoming expensive due to excessive cost of transportation from sources. Also large scale depletion of sources creates environmental problems. Unfortunately, production of cement also involves large amount of carbon dioxide gas into the atmosphere, a major contributor for greenhouse effect and the global warming. To overcome these problems there is a need of cost effective, alternative and innovative materials. These materials are stone quarry dust, fly ash, silica fume, rice husk, recycled waste aggregate , neem seed husk ash ,saw dust etc. some of them are industrial by products and are substantially available. Like fine aggregate much comprehensive effort is made to partially replace the cement by alternative materials .In these inclusive efforts various properties of both fresh and hardened mortar are studied. An experimental study was conducted on the nature of SF (silica fume) and its influences on the compressive strength of mortar. Ultimate Compressive strength has been determined for different mix combinations of materials and these values are compared with the corresponding values of conventional concrete. The author concluded that 30% silica fume gives higher compressive strength (S.F.U.Ahmed et al., 1999). Investigation on strength development of mortar containing limestone was performed. Specimens were placed in prismatic moulds of 40 x 40 x 160 mm in size. The compressive strength decreases with the increase of the content of limestone fines at the age of 90 days. The decrease in strength at long term may be attributed to the dilution effect (B. Menadi et al., 2009).

In this present work the main objective is to determine the acceptability of stone dust as replacing substance of both binding material and fine aggregate in mortar in respect of the normal strength.

2. MATERIALS AND METHOD

2.1 Materials

2.1.1 Cement

In production of mortar, Portland composite cement was used in this study. The properties of cement used in the investigation are presented in Table 1.

Table 1: Properties of cement used

Physical properties			Chemical composition (%)	
Initial setting time	28 Days Compressive strength	Specific surface area(cm2 /gm)	Na_2O	0.170
			MgO	1.860
			Al_2O_3	5.810
			SiO_2	21.02
			P_2O	0.125
60 min	42.5 Mpa	3887	SO_3	1.70
			K_2O	0.470
			CaO	60.88
			TiO_2	0.207
			Fe_2O_3	3.079

2.1.2 Water

Drinkable tap water was used in this study.

2.1.3 Fine aggregate

Sand must be free of impurities, such as salts, clay or other foreign materials. The three key characteristics of sand are particle shape, gradation and void ratio.. In this research wok, two types of sands were used. Sand having fineness modulus 2.74 and 1.22 was used as fine aggregate.

2.1.4 Stone dust

Stone dust was processed in two forms one for replacement of sand and another for cement replacement. For sand replacement the gradation and fineness modulus of stone dust was kept similar to the sand. Stone dust passing by No.200 sieve, was used as cement replacing material. The specific surface area of this dust is 2452 cm^2/gm.

2.2 Method

2.2.1 Processing of stone dust

Stone dust was processed in two forms one for replacement of sand and another for cement replacement. For sand replacement the gradation and fineness modulus of stone dust was kept similar to the sand. At first stone dust was taken under a successive process to make it appropriate for the replacement of both the cement and sand stone dust with moisture was dried in oven at 1050C for 24 hours. After drying screening process was performed. The portion of dust which was retained by no.100 sieve was separated to substitute sand and the remaining part of dust was poured into abrasion machine for making it finer. After the collection of dust from the machine it was screened again by No.200 sieve and dusts passing it were stored as the cement replacing material. The specific surface area of this dust is 2452 cm^2/gm. Sieve analysis for sand and gradation of stone powder was also examined. The testing procedures are summarized in this section.

2.2.2 Sieve analysis of fine aggregate

Determination of fineness modulus of fine aggregate was done according to ASTM C136 which is shown in figure 1.

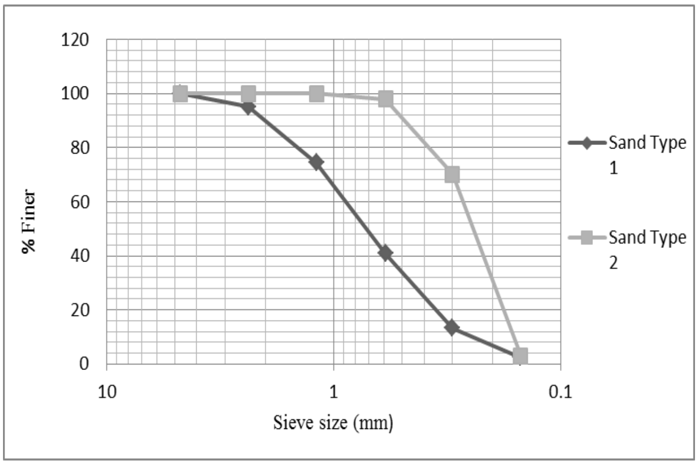

Figure 1: Sieve analysis of fine aggregate

2.2.3 Gradation of stone powder

The mean diameter of this powder form was found 46.6 µm. The Blaine specific surface area was found 2452 cm^2/gm. The blaine specific surface area for cement was found 3884 cm^2/gm.

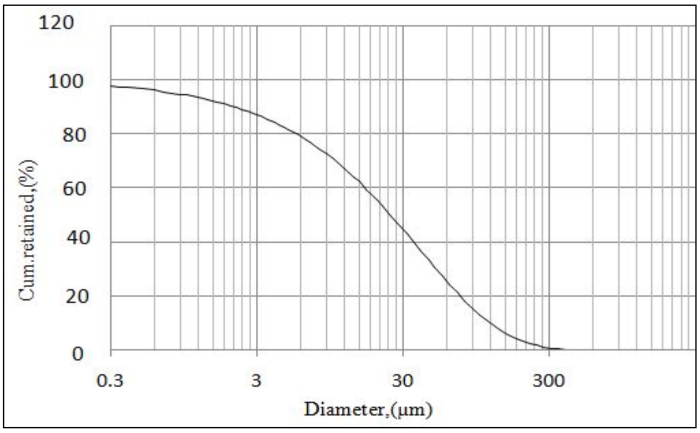

Figure 2: Gradation of stone powder

2.2.4 Mix designation

Two types of sand (Sand S and Sand K) was used in this work. For sand S fineness modulus value is 2.74 and for K value is 1.2. Two different Cement -Sand ratio was used also. They are –A (1:2.75) and B (1:4.50). The mix designation is shown in Table 2.

Table 2: Mix designation and water/cement ratio

Batch	Sand S (kg/m^3)	Sand K (kg/m^3)	Stone Dust (kg/m^3)	Cement (kg/m^3)	Stone Powder (kg/m^3)	W - C Ratio
AS_1	1690	_	_	614	_	0.32
AS_2	1267	_	423	614	_	0.34
AS_3	845	_	845	614	_	0.38
AS_4	1690	_	_	583	31	0.35
AS_5	1267	_	423	583	31	0.37
AS_6	845	_	845	583	31	0.41
AS_7	1690	_	_	553	61	0.36
AS_8	1267	_	423	553	61	0.39
AS_9	845	_	845	553	61	0.40
BS_1	1970	_	_	438	_	0.60
BS_2	1478	_	492	438	_	0.70
BS_3	985	_	985	438	_	0.70
BS_4	1970	_	_	417	21	0.72
BS_5	1478	_	492	417	21	0.70
BS_6	985	_	985	417	21	0.72
BS_7	1970	_	_	395	43	0.74
BS_8	1478	_	492	395	43	0.72
BS_9	985	_	985	395	43	0.74
AK_1	_	1690	_	614	_	0.59
AK_2	_	1267	423	614	_	0.63
AK_3	_	845	845	614	_	0.61
AK_4	_	1690	_	583	31	0.61
AK_5	_	1267	423	583	31	0.61
AK_6	_	845	845	583	31	0.62
AK_7	_	1690	_	553	61	0.61
AK_8	_	1267	43	553	61	0.61
AK_9	_	845	845	553	61	0.62

2.2.5 *Sample preparation*

Two types of sample were prepared as ASTM C 109. They are

(1) Cube samples

Weighted binder and fine aggregate was mixed in the dry condition first, till the mixture seems to be homogenous. Calculated amount of water (from the water-cement ratio) was added to the mixture. Cube moulds were filled on two times periods and in each period tamping was done for 32 times. For each variation about 12 samples were casted and left for curing in clean water tank for 3,7, 28 and 90 days.

(2) Briquette samples

Briquette molds were filled with the same mixture and removed from the mold after 24 hours and left for curing in clean water tank for 3,7, 28 and 90 days.

2.3 Experimental program

Tests were performed on each mortar mix.

2.3.1 *Flow test*

Water-Cement ratio should suitably determine to give the required design strength. In this work water – cement ratio for each of the 27 nos. mix variation was determined According to ASTM Designation: C 109, C 185, C 230. The total mixing variation and water/cement ratio is also shown in table 2.

2.3.2 Compressive Strength test

Compressive strength test of 2 inch cube specimen was performed according to ASTM C 109. Cube specimens were tested at 3, 7, 28 and 90 days using compression Testing Machine at a constant loading rate. The maximum strength of each specimen was recorded and the average of three samples was considered the compressive strength at the specific day.

2.3.3 Tensile strength test

Tensile strength test of briquette sample was performed at 28 and 90 days. The test was performed according to ASTM C 150.

3. RESULTS AND DISCUSSIONS

3.1 Compressive Strength Test

The results are shown in the following figure.

3.1.1 Results for cement -sand ratio 1:2.75 (A) and sand of Fineness modulus 2.74 (S)

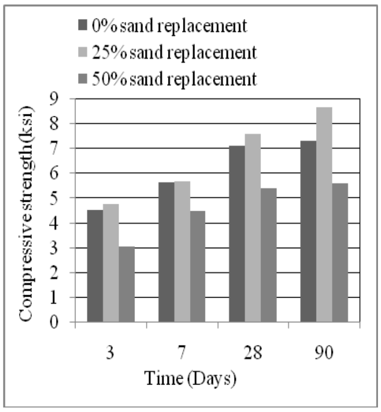

Figure 3.33: Compressive strength for 0% cement replacement

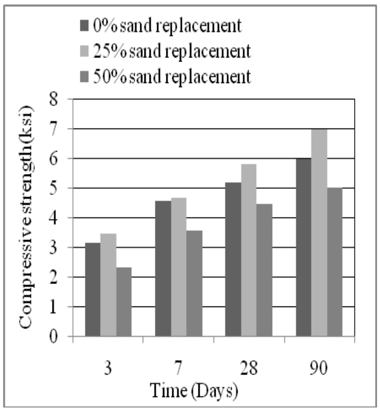

Figure 3.2: Compressive strength for 5% cement replacement

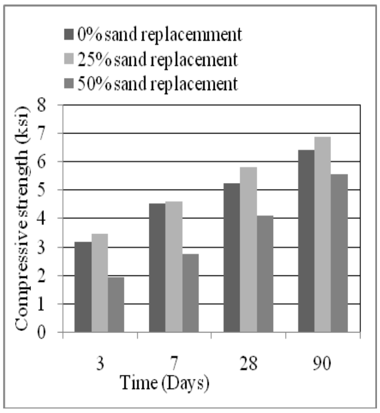

Figure 3.3: Compressive strength for 10% cement replacement

Figure 3-1 reflecting that sand replacement by stone dust up to a certain level shows higher compressive strength value. The highest compressive strength value is 8.682 ksi at the age of 90 days.

In Figure 3-2 the compressive strength values are showing a decreasing trend with the incorporation of stone powder as cement replacing material. That is why the strength value of Figure 3.2 is lower than the values of Figure 3-1. Maximum compressive strength values at the age of 3, 7, 28 and 90 days are 3.483, 4.675, 5.787 and 6.971 ksi respectively.

In Figure 3-3, the values are reflecting an increasing trend up to 25% sand replacement by stone dust. Beyond this replacement level values are decreasing. At the same time effect of cement replacement can also be noticed here. At the age of 90 days the strength value for the samples with 100% sand is 6.396 ksi. With the sand replacement of 25%, 90 days compressive strength value is found 6.871 ksi.

3.1.2 Results for cement –sand ratio 1:4.50 (B) and sand of Fineness modulus 2.74 (S)

In Figure 3.4, the maximum value is 3.033 ksi. At the age of 28 days 0%, 25% and 50% sand replacement shows strength value of 1.742, 2.108 and 1.925 ksi respectively. Compressive strength value is showing an increasing trend with the increase of curing period. For 5% cement replacement the compressive strength values decrease but it is very less.

With the replacement of cement compressive strength is getting reduced. Comparison between Figure 3.4, Figure 3.5 and Figure 3.6 is giving clear insight about the strength reducing trend. In Figure 3.5 and Figure 3.6, at the age of 90 days samples of 25% sand replacement shows strength of 2.81 and 2.533 ksi respectively. But in Figure 3.4, 25% sand replaced samples shows strength value of 3.033 ksi, which is the highest value of the three strength values.

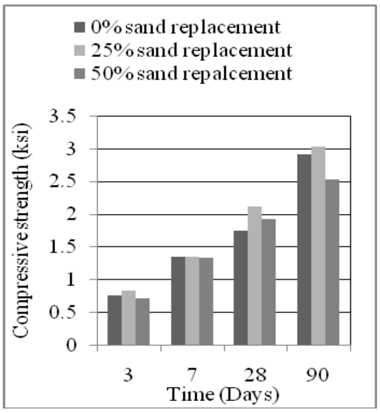

Figure 3.4: Compressive strength for 0% cement replacement

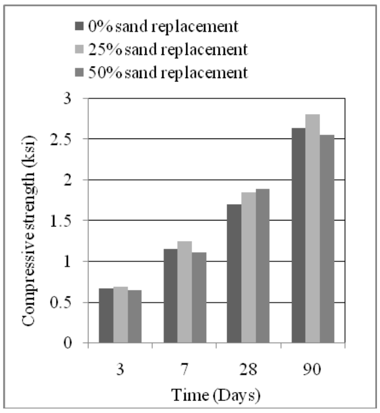

Figure 3.5: Compressive strength for 5% cement replacement

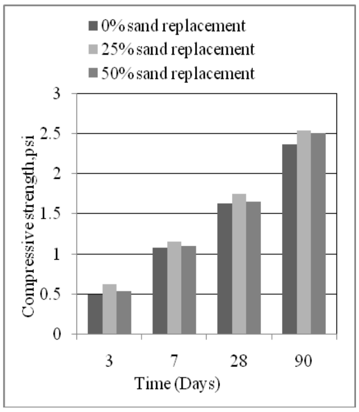

Figure 3.6: Compressive strength for 10% cement replacement

3.1.3 Results for cement –sand ratio 1: 2.75 (B) and sand of Fineness modulus 1.2 (K)

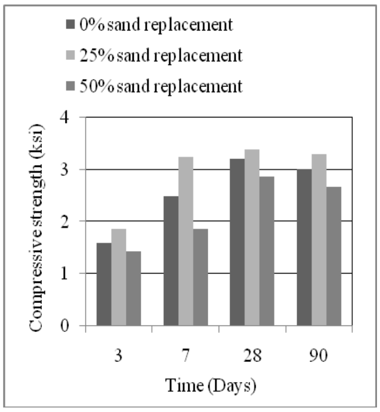

Figure 3.7: Compressive strength for 0% cement replacement

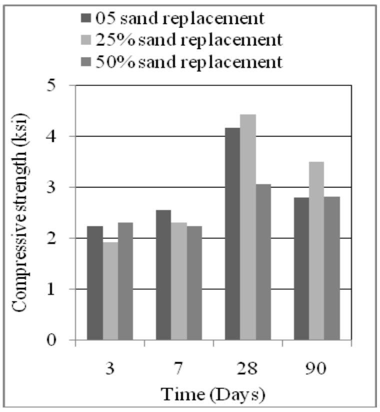

Figure 3.8: Compressive strength for 0% cement replacement

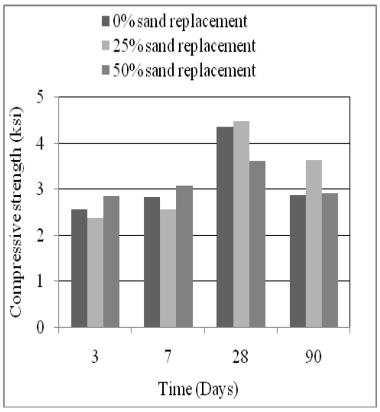

Figure 3.9: Compressive strength for 0% cement replacement

In Figure 3.7, 3.8 and 3.9 the compressive strength values are showing reducing criteria at long term. In Figure 3.7, the strength values of the samples of 0% sand replacement at the age of 28 and 90 days are 3.192 ksi and 3.000 ksi respectively. Also in case of 25% sand replacement, a reduction in strength is noticed. Figure 3.8, depicting the cement replacement effect as well as long term strength reduction.

Cement replacement by stone powder shows increasing trend in case of finer sand (K). In Figure 3.9, 28 and 90 days strength for 50% sand replacement samples are 3.616 and 2.9 ksi respectively.

It was observed that there is reduction in compressive strength with addition of stone dust. In the case of samples prepared with pure cement, when sand is replaced by 25%, the strength value increases. The reason behind this, the presence of dust gives enough particles to fill the voids between cement and aggregate. But with further increment of stone dust the strength value decreased. These phenomena can be explained in terms of water-cement ratio. Both for 25% and 50% replacement level water-cement ratio increase. Because water absorption quality of stone dust is 1-1.5% (Continual Improvement Program, Phase-I Vol.: VIII) more compared to natural sand. This additional water content helps in maintaining the workability .Also this extra water being the absorbed water will not be available for hydration in full and thus do not reduce strength of mortar. But when more stone dust is adding, the extra water for this portion will be available in hydration, thus decrease in strength is occurred for 50% replacement of sand.

3.2 Tensile Strength Test

In this part test *results* obtained for tensile strength is shown.

3.2.1 Results for cement – sand ratio 1:2.75 (A) and sand of Fineness modulus 2.74 (S)

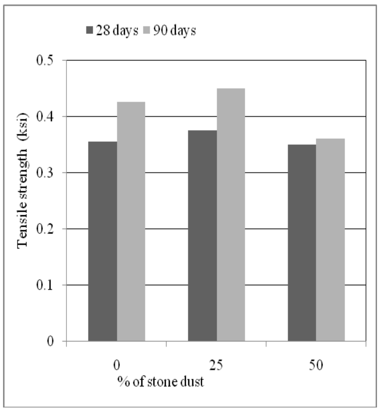

Figure 3.10: Tensile strength of 0% cement replacement

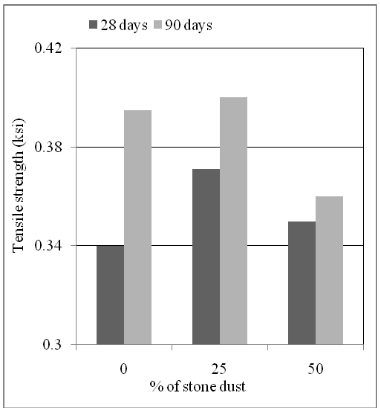

Figure 3.11: Tensile strength of 5% cement replacement

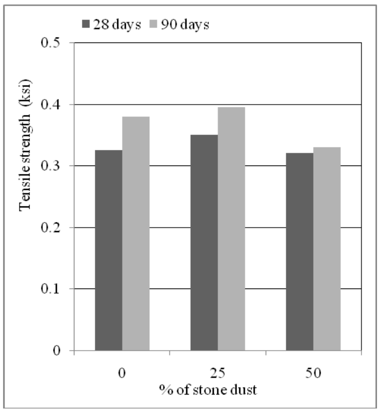

Figure 3.12: Tensile strength of 10% cement replacement

In Figure 3.10, it is presenting that sand replacement by stone dust up to a certain level shows higher tensile strength value than the samples of 100% sand 25% replacement of sand shows highest compressive strength value 0.45 ksi at the age of 90 days.

In Figure 3.11 the compressive strength value decrease which is negligible. In Figure 3.12 the strength value is uniformly less than Figure 3.11. But for all cases the compressive strength increase with the increase of replacement of stone dust up to 25%. Beyond this increase, the strength decreases.

3.2.2 Results for cement – sand ratio 1: 4.50 (B) and sand of Fineness modulus 2.74 (S)

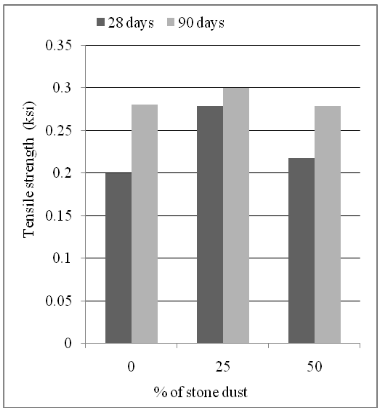

Figure 3.13: Tensile strength of 0% cement replacement

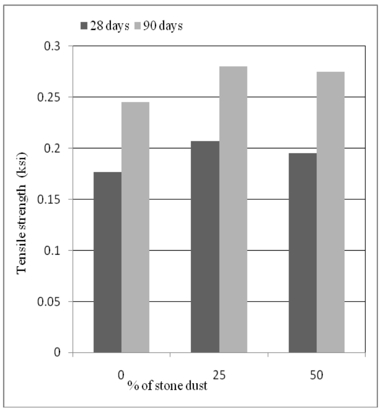

Figure 3.14: Tensile strength of 5% cement replacement

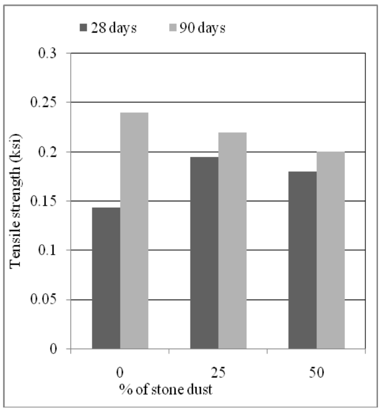

Figure 3.15: Tensile strength of 10% cement replacement

In Figure 3.13, the maximum value is 0.3 ksi. At the age of 28 days 0%, 25% and 50% sand replacement shows strength value of 0.2, 0.278 and 0.217 ksi respectively. Tensile strength value is showing an increasing trend with the increase of curing period.

With replacement of cement by stone powder, reduction in tensile strength is obtained. For this reason value of Figure 3.14, is lower than the value of Figure 3.13.The maximum value of Figure 3.14 is 0.28 ksi. In Figure 3.15 the compressive strength value decrease due to the 10%, cement replacement.

3.2.3 Results for cement – sand ratio 1:2.75 (A) and sand of Fineness modulus 1.2 (K)

In Figure 3.16, Figure 3.17 and Figure 3.18 the tensile strength value of briquette samples are shown. The strength value is increasing with 25% sand replacement by stone dust. When cement replacement occurring the strength value shows a decreasing trend. There are many factors behind such type of variation in results. Main cause is the particle size. The particle size of stone powder is greater than the cement particles. Mean particle size of stone power and cement is 24 and 46 μm respectively, That is why with the replacement of cement by stone powder reduce the available surface area involved in the hydration reaction of cement.

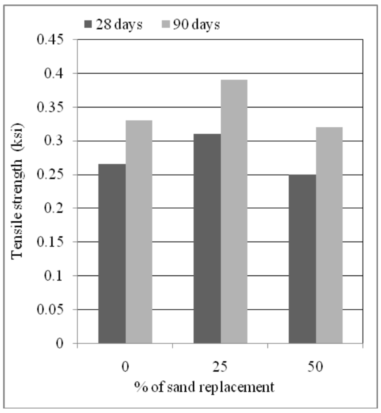

Figure 3.16: Tensile strength of 0% cement replacement

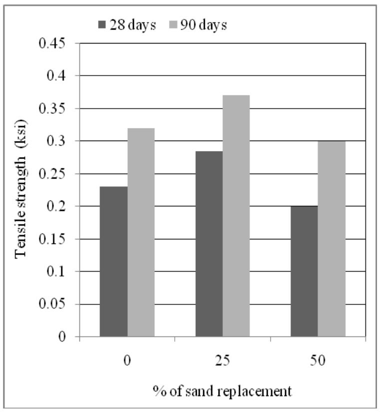

Figure 3.17: Tensile strength of 5% cement replacement

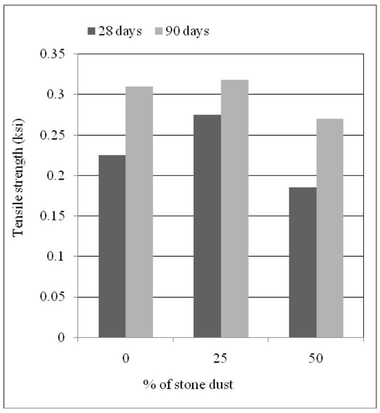

Figure 3.18: Tensile strength of 10% cement replacement

4. CONCLUSIONS

[1] In case of cement-sand ratio 1:2.75 (A) and 1:4.50 (B), and sand of Fineness modulus 2.74 (S), the compressive strength increases with the increase of sand replacement by stone powder up to 25% . Beyond this value compressive strength decreases. On the other hand increase of cement replacement reduces the compressive strength. Maximum compressive strength was found 8.682 ksi and 3.033 ksi for cement-sand ratio of A and B respectively.

[2] For cement-sand ratio 1:2.75 (A) and sand of Fineness modulus 1.2 (K), compressive strength values are showing reducing criteria at long term i.e. after 28 days curing strength reduces. The maximum value is 4.459 ksi for 25% sand and 10% cement replacement.

[3] In cement-sand ratio 1:2.75 (A) and 1:4.50 (B), and sand of Fineness modulus 2.74 (S), tensile strength increases up to 25% replacement of sand. 25% sand and 0% cement replacement produces the maximum tensile strength. Maximum tensile strength is 0.45 ksi and 0.3 ksi for water-cement ratio A and B.

[4] For cement-sand ratio 1:2.75 (A) and sand of Fineness modulus 1.2 (K), tensile strength increases up to 25% replacement of sand. Beyond this value tensile strength decreases. Maximum strength is 0.39 ksi. which was found for 25% sand and 0% cement replacement.

[5] Samples of AS variation (0% cement replacement) provides 90 days strength of 0% and 25% sand replacement are 7.320 and 8.682 ksi respectively. For the samples of BS variation (0% sand replacement) of 0% and 5% cement replacement are respectively 2.906 and 2.640 respectively. For 0% cement and 25% sand replacement, at the age of 28 days the strength of samples of AS and AK variation are 7.584 and 3.379 ksi respectively.

[6] The Compressive Strength of mortar is always maximum for 25% replacement level of sand by stone dust. Replacement of cement by stone powder reduces the strength value of mortar. Coarser sand (S) shows higher strength value than the finer one (K). Mortar mixes manufactured with coarse sand exhibit a more deformable and ductile behaviour.

REFERENCES

A. A. Raheem, B. S. Olasunkanmi and C. S. Folorunso. (2012). "Saw Dust Ash as Partial Replacement for Cement in Concrete". Organization, Technology and Management in Construction, Vol.4, pp. 474-480.

Aggregates for Concrete, ACI Education Bulletin E1-07, developed by ACI Committee E-701, chaired by David M. Suchorski, American Concrete Institute 2007.

Abdul Hakeem Hamed Ahmed, Rana Burhan Abdurrahman and Zena Adel Mohammed. (2009). "Influence of Limestone Powder as Partial Replacement of Cement on Concrete and the Effect of High Temperature on It". Civil Engineering Department, University of Mosul, pp. 24-34.

AmarnathYerramala, Rama Chandurdu C and Bhaskar Desai V. (2012). "Influence of Fly Ash Replacement on Strength Properties of Cement Mortar", International Journal of Engineering Science Technology (IJEST), Vol.4, pp. 3657-3665.

A. Sivakumar and Prakash M. (2011). "Characteristic studies on the mechanical properties of quarry dust addition in conventional concrete". Journal of Civil Engineering and Construction Technology, Vol. 2(10), pp. 218-235.

B. Menadi, S. Kenai, J. M. Khatib and A. Ait-Mokhtar. (2009). "Strength Development and Prediction of Mortars Containing Limestone Fines". SBEIDCO –1st International Conference on Sustainable Built Environement Infrastructures in Developing Countries, ENSET Oran (Algeria), pp. 91-98.

Er. Lakhan Nagpal, Arvind Dewangan, Er. Sandeep Dhiman and Er. Sumit Kumar. (2013). "Evaluation of Strength Characteristics of Concrete Using Crushed Stone Dust as Fine Aggregate". International Journal of Innovative Technology and Exploring Engineering (IJITEE), Vol.2, pp. 102-104.

J. Sridhar and R. Malathy. (2011). Study on Compressive Strength of Cement Mortar with Partial Replacement of Fine Aggregate by Steel Slag for Ferrocement Laminates". International Journal of Earth Sciences and Engineering, Vol. 04, pp. 1139-1144.

Lohani T.K., Padhi M., Dash K.P.and Jena S. (2012). "Optimum utilization of Quarry dust as partial replacement of sand in Concrete". Int. Journal of Applied Sciences and Engineering Research, Vol. 1, pp. 391-404.

Layla Muhsan Hasan Bdeir. (2012). "Study Some Mechanical Properties of Mortar with Sawdust as a Partially Replacement of Sand". Anbar Journal for Engineering Sciences, Vol.5, pp. 22-30.

Mohaiminul Haque, Sourav Ray and H. M. A. Mahzuz. (2012). "Use of Stone Powder with Sand in Concrete and Mortar: A Waste Utilization Approach". ARPN Journal of Science and Technology, VOL. 2, pp. 613-618.

Mohammed Nadeem and Dr. A. D. Pofale. (2012). "Replacement of Natural Fine Aggregate With Granular Slag - A Waste Industrial By-Product In Cement Mortar Applications As An Alternative Construction Materials''. International Journal of Engineering Research and Applications (IJERA), Vol. 2, pp.1258-1264.

M. Shariq, J. Prasad and A.K. Ahuja (2008). "Strength Development of Cement Mortar and Concrete Incorporating GGBFS". Asian Journal of Civil Engineering (BUILDING AND HOUSING), VOL. 9, pp. 62-74.

M.R. Karim, M.F.M. Zain, M. Jamil, F.C. Lai and M.N. Islam. (2012). "Strength of Mortar and Concrete as Influenced by Rice Husk Ash: A Review". World Applied Sciences Journal, Vol.10, pp.1501-1513.

M. Bai, D. Niu, J. Shang and X. Wu. (2008). "Experiment research on cement mortar with steel slag as fine aggregate". International Conference on Microstructure Related Durability of Cementitious Composites, pp. 1283 – 1290

Nagabhushana and H. Sharada bai. (2011). "Use of crushed rock powder as replacement of fine aggregate in mortar and concrete". Indian Journal of Science and Technology, Vol. 4 , pp. 917-922.

P.A. Shirule, Ataur Rahman and Rakesh D. Gupta. (2012). "Partial Replacement of Cement With Marble Dust Powder". International Journal of Advanced Engineering Research and Studies, Vol.1, pp.175-177.

Radhikesh P. Nanda,Amiya K. Dasand Moharana.N.C.(2010)." Stone crusher dustas a fine aggregate in Concre te for paving blocks". International Journal of Civil and Structural Engineering, Vol. 1, pp.614-620.

R. Ilangovana, N. Mahendrana and K. Nagamanib. (2008). "Strength and Durability Properties of Concrete Containing Quarry Rock Dust as Fine Aggregate". ARPN Journal of Engineering and Applied Sciences, VOL. 3, pp. 20-26.

Shaikh Faiz Uddin Ahmed, Yoshihiko Ohama and Katsunory Demura. (1999). "Comparison of Mechanical Properties and Durability of Mortar Modified by Silica Fume and Finely Granulated Blast Furnace slag". Journal of Civil Engineering, The Institution of Engineers, Bangladesh, Vol. CE 27,pp. 144-154.

S. Sorlini, L. Rondi, C. Collivignarelli and A. Abba. (2012). "Steel Slag Characterization as Aggregate in Concrete". International Conference on Industrial and Hazardous Waste Management.

T. Hoque, M. H. Rashid, MD. R. Hasan, E. F. Mondol (2013). "Influence of Stone Dust as Partially Replacing Material of Cement and Sand on some Mechanical Properties of Mortar". International Journal of Advanced Structures and Geotechnical Engineering , ISSN 2319-5347, Vol. 02, No. 02, April 2013

V. Syam Prakash, Dhanya Krishnan and G. Jeenu, 2007, "Influence of Fine Stone Dust on High Strength Concrete", Singapore Concrete Institute, Article Online Id: 100032054.

ANALYSIS OF FACTORS CAUSING SCHEDULE DELAY IN CONSTRUCTION PROJECTS IN BANGLADESH

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ABSTRACT

The construction industry is one of the key sectors that provide important elements for the development of a country's economy. Thus, it has been needed to efficient sound management for its development. However, many construction projects have faced with various types of problems. In such problems, delay is the most common problem in the construction industry. It has many negative effects on project's success in terms of time, cost, quality, and safety. From the literature review, a total of thirty-five delay factors were selected and were divided into seven groups. This study was carried out to identify the main causes of delay through their importance level for a construction project. The importance level was determined based on the frequency of occurrence and severity of impact. The structured questionnaire has distributed to the respondents who have much experience in construction and management in Bangladesh. The results of analysis indicated that seven top factors of construction delay according to their level of importance. It is hoped that the finding of this study is the significant contribution to the current performance of project in controlling time overruns.

Keywords: Factors of delay, construction industry, frequency index, severity index, importance index, Bangladesh.

1. INTRODUCTION

Bangladesh is one of the newly born developing countries. It has one of the highest population densities countries in the Globe. More than 1000 peoples live per Square Kilometer (Kamruzaman et al., 2012). Bangladesh is ranked as the 195th country according to the Gross Domestic Product (GDP). Construction industry has a significant role in contributing to the overall development of a country. Bangladesh construction industry has been growing fast in recent years. During 2008-2009, total employment of construction sector stood at 2.024 million. Even at the current growth rate, the total employment in the construction sector is expected to be 2.88 million by 2014 and 3.32 million by 2020, (Shaon, 2012). Therefore, it has been needed to efficient sound management for the quick development of construction industry as well as national economic growth. Delay in construction work is severe in our country and affecting almost every construction project. Due to construction delay, our national economy as well as infrastructure development is affected vigorously. Therefore it is very important to find out the actual reason of construction delay and its management practice.

The construction industry is one of the key sectors that provide important elements for the development of a country's economy. Thus, it has been needed to efficient sound management for its development. However, many construction projects have faced with vigorously various types of problems. In such problems, delay is the most common problem in the construction industry. It is being caused a multitude of negative effects on the project and its participating parties. It is influenced by several factors which are mentioned in many previous studies. The top most delay factors is presented from previous different studies such as (i) lack of commitment, (ii) insufficient of site management (iii) poor site coordination (iv) improper planning (v) lake of clarity in project scope for Indian construction projects, (Doloi et al., 2012); (i) delay in decision making, (ii) delay in providing permanent utilities, (iii) change of scope, (iv) payments delay for civil engineering projects in Middle-East in Egypt, (Ezeldin et al., 2013); (i) financing by contractor during construction, (ii) delays in contractor's payment by owner, (iii) design changes by owner or his agent during construction, (iv) partial payments during construction (v) Slow delivery of materials for building construction projects in Egypt, (El-Razek et al., 2008); (i) poor site management and supervision, (ii) poor project management assistance, (iii) financial difficulties of owner, (iv) financial difficulties of contractor for large construction projects in Vietnam, (Long et al., 2008), and (i) political situation, (ii) limited working area, (iii) award project to lowest bid price, (iv) progress payments delay by owner, (v) delays in decision making by owner for road construction projects in Palestine, (Ibrahim et al., 2012), and so on. It has been seen that factors influenced delays are different with location and type of construction. Delays in construction are very costly for the most part and completing project on time is beneficiary to all project's parties, (El-Razek et al., 2008). Thus, it is needed to predict perfectly the delay factors by ranked according to the most responsible for project delays and should take preventive measure in the early stage of construction. The success of construction projects is depending on efficient management. The main purpose of this study is to identify the main causes of delay through their importance level for a construction project.

This study has been conducted to investigate the main reasons for construction delay and find way to overcome them. It will be also helpful to reduce construction time and undue cost which are almost common problem in every construction work.

2. LITERATURE REVIEW

A number of articles and studies have been carried out to identify the causes of delay in construction projects, in internationally but locally not much that means in Bangladesh. Baldwin and Manthei (1971) investigated the fundamental causes of delay in building construction projects in the United States. They surveyed among engineers, architects, and contractors, and concluded that there was substantial agreement among the three groups regarding the causes of delay. They also revealed that weather, labor supply, and subcontractors were the major causes of delay.

Assaf et al. (1995) discussed a survey research and pointed out 56 main causes of delay in large building construction projects. These caused were gathered into nine separated major groups: materials, manpower, equipment, financing, environment, changes, government relations, contractual relationships, and scheduling and controlling techniques with different levels of frequency and severity to different parties. Furthermore, Kaming et al. (1997) studied influencing factors on 31 high-rise projects in Indonesia and found out that cost overruns occur more frequently and are more severe problem than time overruns. They pointed out that the major factors influencing cost overrun are material cost increase due to inflation, inaccurate material estimation and degree of complexity. While in time overrun, the most important factors causing delays are design changes, poor labor productivity, inadequate planning, and resource shortages.

Chan and Kumaraswamy (1997) carried out a survey to evaluate the relative importance of 83 potential delay factors in Hong Kong construction projects and found five most important factors: poor risk management and supervision, unforeseen site conditions, slow decision making, client-initiated variations, and work variations. Similarly, Al-Momani (2000) investigated causes of delay in 130 public projects in Jordan. The main causes of delay were related to designer, user changes, weather, site conditions, late deliveries, economic conditions and increase in quantity. The study suggested that special attention to factors will help industry practitioners in minimizing contract disputes. Delays have strong relationship with failure and ineffective performance of contractors from Sadi and Sadiq (2006).

El-Razek (2008) et al. investigated to identify the main causes of delay in construction projects from the point of view of contractors, consultants, and owners in Egypt. Their overall results indicated that the most important influential causes are: financing by contractor during construction, delays in contractor's payment by owner, design changes by owner or his agent during construction, partial payments during construction, and non-utilization of professional construction/contractual management. In the same line of study, Ibrahim et al. (2012) discussed a study to investigate the time performance of road construction projects to identify the causes of delay and their severity according to contractors and consultants through a questionnaire survey in the West Bank in Palestine. They identified totally 52 causes of delay during their research. The survey concluded that the top five severe delay causes are political situation, segmentation of the West Bank and limited movement between areas, award project to lowest bid price.

Ezeldin and Ghany (2013) conducted a study in three phases to identify and rank the major causes of delays and to determine the party responsible for the main causes of delays for engineering projects. The results revealed that the causes of delays can be grouped into five (5) main categories: (i) construction related causes (ii) managerial related causes (iii) political related causes (iv) financial related causes (v) technical related causes. They identified top influential 12 causes included 3 construction, 7 managerial, 1 political and 1 financial related cause. Third party is responsible for the remaining two causes.

Akogbe et al. (2013) analyzed of delay factors which affect the delay in construction completion in Benin. This study was identified top ten important delay factors by their rank as (i) Financial capability by contractor, (ii) financial difficulties by owner, (iii) poor subcontractor performance, (iv) materials procurement of contractor, (v) changes in drawings of architect, (vi) inadequate planning and scheduling of contractor, (vii) slow inspection of completed works by the consultant, (viii) equipment availability of contractor, (ix) preparation and approval of drawings of consultant, and (x) acceptance of inadequate design drawings by consultant.

3. RESEARCH METHODOLOGY

This study was undertaken in two phases. The first phase included an extensive literature review and interviews of construction's practitioners regarding questionnaire design who have much experience in construction and management sector. Through the literature review, some factors of delay were selected, some were merged and deleted, and some factors of delay were found in Bangladesh perspective. Finally 35 delay factors were selected to meet the objectives of this study. In the second phase included a questionnaire was developed using the delay causes shows in Table 1.

3.1 Questionnaire Design

Data were gathered through a set of questionnaire survey. The questionnaire of this research work was mainly designed to collect data related to factors of construction delay and project's information. The questionnaire was divided into two main parts. Part one is related to the list of the identified factors of delay in construction project. It was divided into 07 (seven) groups according to the source of delay. Factors are related to Materials, Manpower and equipment, Owner, Consultant, Contractor, Project, and External groups. Second part of questionnaire is included the general information for both the project and respondent. The respondents were requested to answer the questions pertaining to their experience in the construction industry; are related to frequency and severity of construction delay factor's occurrence according to the scale such as '0 = No; 1 = Rarely; 2 = Sometimes; 3 = Often; 4 = Always; for Frequency; and '0 = No; 1 = little; 2 = Moderate 3 = Very; 4 = Extremely; for Severity. For each cause two questions were asked: What is the frequency of occurrence for this cause? And what is the degree of severity of this cause on project delay? Both frequency of occurrence and severity were categorized on a five-point scale is mentioned above.

3.2 Data Collection and Analysis Approach

A set of questionnaire survey was used to collect data for the frequency and severity occurrence of factors of delay responsible for construction project in Bangladesh. The methods of sampling are particular target sampling and random sampling was selected for this study. A targeted sampling is a sampling of the subject matter, be that people or other things is selected using certain criteria. Random sampling is a sampling of a population in which the population is first divided into distinct subpopulations, or strata, and random samples are then taken separately from each stratum. The method of delivering the questionnaire was mainly based on electronic mail. The respondents were selected from the catalogue of REHAB (The Real Estate and Housing Association of Bangladesh) and IEB (Institution of Engineers Bangladesh) and other reliable sources. In addition to increasing the representativeness of samples, stratified random sampling was a useful technique that made general statements about the portions of the population Love et al. (2005). After eliminating the uncompleted questionnaires, 59 data sets were found to be usable in this study.

In a total of 59 data sets, majority (42.37%) of them were collected from building and residential projects, and others data were collected from civil projects, industrial projects, and others type of construction projects which were completed in between 2005 and 2013 are shown in Fig. 1. Regarding respondent's position of work, majority (38.98%) of them is consultant; whereas 33.90%, 11.86%, and 15.25% are owner, contractor, and others respondents respectively (see Fig. 2). Furthermore, regarding respondent's involvement in projects, majorities' respondents (57.63%) were involved in more than four projects, and others were involved in 1 project, 2 projects, 3 projects, and 4 projects are shown in Fig. 3. In addition. respondent's working experience, 30.51% of total respondents have more 12 years, and 30.51% have 8~12 years working experience in construction project, and rest of respondents have 4~8 years and less than 4 years working experience (see Fig. 4). Finally, regarding the total cost of the project, 25 projects are the small projects whose total investment below BDT 500 million, 18 projects are the medium projects whose total investment between BDT 500 million and 2.5 billion, and 18 projects are the large projects whose total investment above BDT 2.5 billion are shown in Fig. 5.

Table 1. Source of Factors of Construction Delay

Group	Factors of Delay	Souce of factors of delay from previous study: Assaf and Al Hejji (2006)	El-Razek et al. (2008)	Ibrahim et at. (2012)	Doli et al. (2012)	Akogbe et al. (2013)	Marzouk and El-Rasas (2013)
Materials	Slow/ lately delivery and shortages of construction materials, F1	√	√	√	√		√
	Price of construction materials increase very rapidly, F2				√	√	
	Damage of materials in storage, F3	√			√		
Manpower and equipment	Shortages of skilled workers, F4	√	√	√		√	√
	Poor labor productivity, F5	√	√	√	√		√
	Shortage of equipment, F6	√	√				
	Lack of equipment efficiency and unskilled operators, F7	√		√	√	√	
	Frequently break down of equipment, F8	√	√				√
Owner	Poor communication by owner with other construction parties, F9	√	√	√	√		√
	Delays in decision making by owner, F10	√	√	√	√	√	√
	Late issuing of approval documents and sample materials by owner, F11	√	√	√	√	√	√
	Delay in running bill payments to contractor and financial difficulties of owner, F12	√	√	√	√	√	√
	Conflict between owners and other parties, F13	√			√		
	Rework due to change of design or deviation order by owner or his agent during construction, F14		√			√	
Consultant	Poor/ deficient planning and estimate, F15					√	√
	Slow examine/ inspection of completed works by consultant, F16	√	√	√		√	
	Delay in design works, F17	√		√			
	Inappropriate design or mistake in design made by designers, F18	√	√	√		√	√
Contractor	Delay in material procurement (action by the contractor), F19	√			√	√	
	Difficulties in financing project by contractor, F20	√	√	√	√		√
	Poor site management and supervision by contractor, F21	√		√	√	√	√
	Inadequate experience of contractor, F22	√	√	√	√		√
	Incompetent/ Immature subcontractor, F23					√	
	Frequent change of subcontractor, F24	√			√		
Project	Changes in types and specifications during construction, F25	√	√	√	√		√
	Change orders during construction, F26	√		√		√	√
	Additional/ increasing in quantity of works, F27			√	√	√	
	Unrealistic project time estimation and imposed in contract, F28		√	√	√		√
	Faulty/ mistake in soil investigation report, F29		√		√		
	Rework because of errors during construction, F30	√			√	√	√
External	Excessive bureaucracy in owner operation, F31		√		√		
	Obtaining permissions from local authorities, F32	√	√		√		√
	Political situation (Revolution/ public strikes), F33	√		√	√		
	Government/ public interruptions, F34	√		√	√	√	√
	Natural disaster, F35	√	√	√	√	√	√

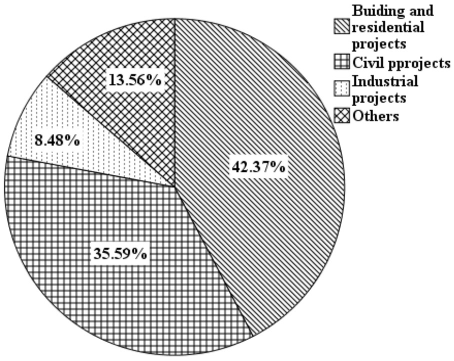

Fig. 1 Project types in survey

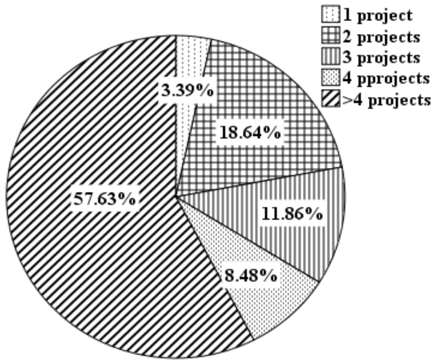

Fig. 3 Respondent’s involvement in project

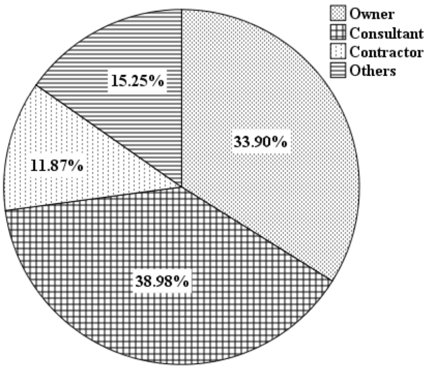

Fig. 2 Project parties in survey

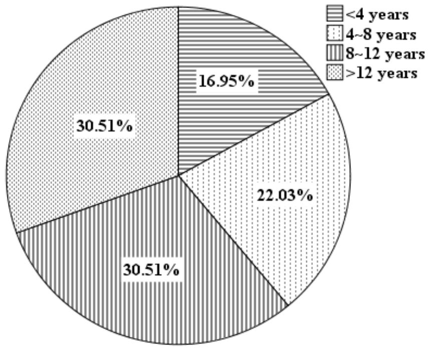

Fig. 4 Respondent’s work experience in survey

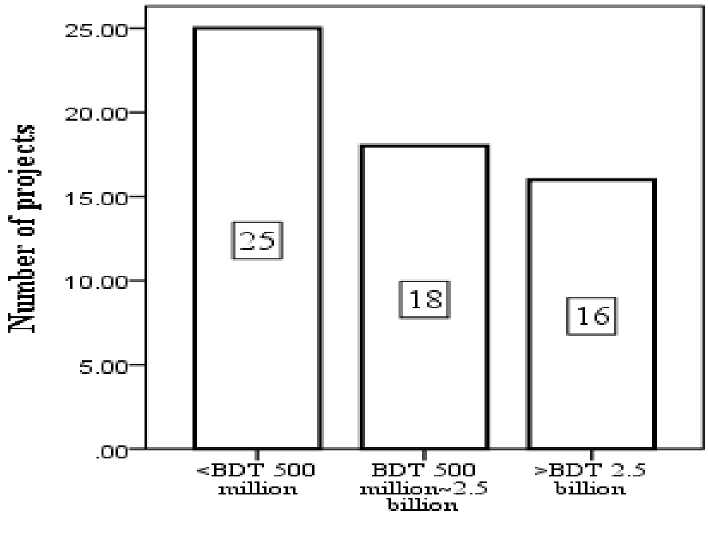

Fig. 5 Project size by investment of project.

3.3 Analysis Tools

The survey data were processed through three types of indices:

• *Frequency index(FI):* is the number of times it happens during a particular period. This index expresses occurrence frequency of factor responsible for delay. It was computed as per following equation 1.

$$FI = \frac{\sum_{0}^{4} a_i n_i}{4N} \quad \text{.................................. (1)}$$

where: a = constant expressing the weight assigned to each responses (ranges from 0 for No happen to 4 for Always), n =frequency of each response, N = total number of responses.

• *Severity index(SI):* is the degree of influence of a factor to the performance.This index expresses severity of factor that caused delay. It was computed as per following equation 2.

$$SI = \frac{\sum_{0}^{4} a_i n_i}{4N} \quad \text{.................................. (2)}$$

where: *a* = constant expressing the weight assigned to each responses (ranges from 0 for *No Severe* to 4 for *Extremely*), *n* =frequency of each response, *N* = total number of responses.

- *Importance index (IMP. I):* This index expresses the overview of factor based on both their frequency and severity. It was computed as per following equation 3.

$$IMP.I = FI \times SI \quad \text{... (3)}$$

3.4 Pearson's rank correlation

Pearson's coefficient of rank correlation is used to demonstrate whether there is the agreement or disagreement among each pair of parties. Table 2 illustrates the results of Pearson's coefficient and significance level calculations. A conclusion can be inferred from these results that there is an almost good agreement between parties. Among all parties, four pair of parties (such as owner-consultant, owner-contractor, owner-others, and consultant-others) were showed that a very good positive agreement in ranking these causes despite frequency, severity or importance index and another two pair of parties, consultant-contractor, and contractor-others were showed positive agreement with lower level. The correlation coefficient varies between +1 and -1, where +1 implies a perfect positive relationship (agreement), while -1 results from a perfect negative relationship (disagreement). It might be said then that sample estimates of correlation close to unity in magnitude imply good correlation, while values near zero indicate little or no correlation. The Pearson's rank correlation coefficient r is also used to measure and compare the association between the rankings of two parties for a single cause of delay, while ignoring the ranking of the third party. And it was calculated by the following equation 4.

$$r = \frac{\sum XY - \frac{(\sum X)(\sum Y)}{n}}{\sqrt{(X^2 - \frac{(\sum X)^2}{n})(\sum Y^2 - \frac{(\sum Y)^2}{n})}} \quad \text{.......................... (4)}$$

where r is the Pearson rank correlation coefficient between two parties, X and Y are the variables of two groups or parties, and n is the number of pairs of rank.

Table 2. Pearson's correlation between parties

Parties	Frequency Index		Severity index		Importance Index	
	Pearson correlation	Sig. level	Pearson correlation	Sig. level	Pearson correlation	Sig. level
Owner-consultant	0.607	0.000	0.343	0.044	0.625	0.000
Owner-contractor	0.279	0.104	0.373	0.027	0.410	0.014
Owner-others	0.535	0.001	0.551	0.001	0.663	0.000
Consultant- contractor	0.276	0.109	0.126	0.471	0.217	0.210
Consultant-others	0.749	0.000	0.461	0.005	0.728	0.000
Contractor--others	0.313	0.067	0.020	0.908	0.261	0.130

4. RESULT AND DISCUSSION OF ANALYSIS

The factors of delay were ranked according to their importance index. The importance level was determined based on the frequency of occurrence and severity of impact. They were calculated by the equations 1 and 2 respectively as shown in Table 4 and Table 5. The factors of delay were arranged in decending order according to their importance index of each factor. They were calculated by the equation 3 as shown in Table 6. The top seven most important causes of delay were identified by the survey, and based on overall results are: (i) price of

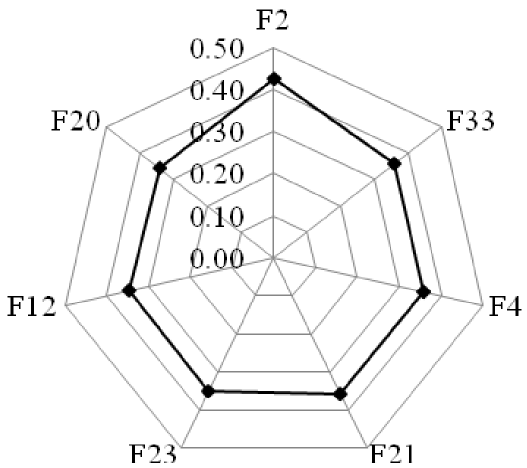

Fig 6. Spider mapping chart of important index of top seven factors of delay for overall cases.

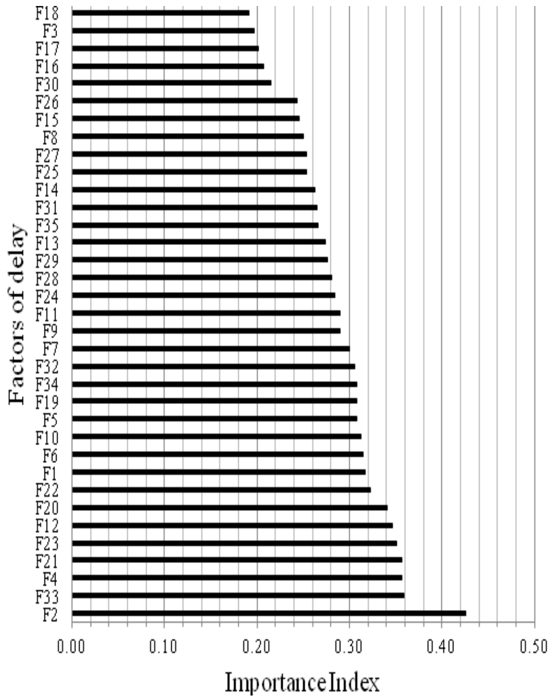

Fig. 7 Importance index of all factors of delay

Table 3. Top seven most important factors of delay by project party

Rank	Overall result	Owner	Consultant	Contractor	Others
1	Price of construction materials increase very rapidly	Price of construction materials increase very rapidly	Price of construction materials increase very rapidly	Shortages of skilled workers	Poor site management and supervision by contractor
2	Political situation (Revolution/ public strikes)	Poor site management and supervision by contractor	Incompetent/ Immature subcontractors	Price of construction materials increase very rapidly	Shortages of skilled workers
3	Shortages of skilled workers	Shortages of skilled workers	Delay in running bill payments to contractor and financial difficulties of owner	Slow/ lately delivery and shortages of construction materials	Price of construction materials increase very rapidly
4	Poor site management and supervision by contractor	Difficulties in financing project by contractor	Shortage of equipment	Poor communication among the construction parties	Incompetent/ Immature subcontractors
5	Incompetent/ Immature subcontractors	Political situation (Revolution/ public strikes)	Poor site management and supervision by contractor	Delays in decision making by owner	Delay in running bill payments to contractor and financial difficulties of owner
6	Delay in running bill payments to contractor and	Inadequate experience of contractor	Difficulties in financing project by contractor	Political situation (Revolution/ public strikes)	Unrealistic project time estimation and imposed in contract

	financial difficulties of owner				
7	Difficulties in financing project by contractor	Delay in material procurement (action by the contractor)	Obtaining permissions from local authorities	Delay in running bill payments to contractor and financial difficulties of owner	Faulty/ mistake in soil investigation report

Table 4. Frequency index and ranking

Factors	Owner		Consultant		Contractor		Others		Overall	
	FI	Rank	FI	Rank	FI	Rank	FI	Rank	FI	Rank
F1	0.525	15	0.576	9	0.607	1	0.667	6	0.572	6
F2	0.650	1	0.630	1	0.536	3	0.778	1	0.648	1
F3	0.463	32	0.402	35	0.393	20	0.417	34	0.419	34
F4	0.588	5	0.554	16	0.607	2	0.722	2	0.597	3
F5	0.563	11	0.554	17	0.500	6	0.639	10	0.564	9
F6	0.575	8	0.609	2	0.393	21	0.611	16	0.568	8
F7	0.500	22	0.543	19	0.464	11	0.583	21	0.525	22
F8	0.525	16	0.500	24	0.429	16	0.500	31	0.500	25
F9	0.575	9	0.478	31	0.536	4	0.583	22	0.534	17
F10	0.475	28	0.565	13	0.500	7	0.722	3	0.547	13
F11	0.475	29	0.598	4	0.464	12	0.639	11	0.547	14
F12	0.563	12	0.598	5	0.500	8	0.722	4	0.589	5
F13	0.475	30	0.576	10	0.500	9	0.583	23	0.534	18
F14	0.475	31	0.500	25	0.500	10	0.611	17	0.508	24
F15	0.488	24	0.500	26	0.286	33	0.556	27	0.479	30
F16	0.450	33	0.467	32	0.357	23	0.583	24	0.462	32
F17	0.488	25	0.457	33	0.429	17	0.528	30	0.470	31
F18	0.400	35	0.446	34	0.250	35	0.472	33	0.411	35
F19	0.525	17	0.565	14	0.393	22	0.639	12	0.542	15
F20	0.588	6	0.576	11	0.321	28	0.611	18	0.551	12
F21	0.650	2	0.609	3	0.321	29	0.694	5	0.597	4
F22	0.563	13	0.587	6	0.429	18	0.611	19	0.564	10
F23	0.600	4	0.576	12	0.357	24	0.667	7	0.572	7
F24	0.500	23	0.489	27	0.464	13	0.556	28	0.500	26
F25	0.488	26	0.489	28	0.321	30	0.583	25	0.483	29
F26	0.488	27	0.533	21	0.286	34	0.611	20	0.500	27
F27	0.525	18	0.543	20	0.321	31	0.667	8	0.525	23
F28	0.513	20	0.554	18	0.357	25	0.667	9	0.534	19
F29	0.538	14	0.565	15	0.321	32	0.639	13	0.534	20
F30	0.450	34	0.489	29	0.429	19	0.306	35	0.441	33
F31	0.575	10	0.533	22	0.357	26	0.556	29	0.530	21
F32	0.513	21	0.587	7	0.464	14	0.639	14	0.555	11
F33	0.650	3	0.587	8	0.536	5	0.639	15	0.610	2
F34	0.588	7	0.511	23	0.464	15	0.583	26	0.542	16
F35	0.525	19	0.489	30	0.357	27	0.500	32	0.487	28

construction materials increase very rapidly, (ii) political situation (revolution/ public strikes), (iii) shortages of skilled workers, (iv) poor site management and supervision by contractor, (v) incompetent/ immature subcontractors, (vi) delay in running bill payments to contractor and financial difficulties of owner, and (vii) difficulties in financing project by contractor shows in Fig. 6. In addition, the importance indices of all the factors of delay that influentially affected the construction projects shows in Fig. 7.

In order to make out how can be mitigated of project delay, it is important to identify the responsible party, factors of groups based on source of delay, and project sizes and so on. Therefore, the responsibility of the delay causes from owner, consultant, contractor and others (such as architects) is illustrated in Table 3. In this case, within the seven influential most important causes, three of the causes are under the contractor group, one under owner's group, one under material group, one under manpower & equipment group, and one factor under external group. It can be concluded that the factors of delay have mixed responsibility, and no single party or group is responsible for construction delay. This means that any step to prevent or mitigate delay has to be a joint attempt and based upon teamwork. A same type of observation was concluded in the study of El Razek et al. (2008) and Abdul-Rahman et al. (2006) which conducted in Egypt and Malaysia respectively.

Results by the overall case analysis, among the top seven factors of delay there are one common cause responsible for all parties is "price of construction materials increase very rapidly," and there are four more common causes between consultant and others party; these causes are: "price of construction materials increase very rapidly," "incompetent/ immature subcontractors," "delay in running bill payments to contractor and financial difficulties of owner," and "poor site management and supervision by contractor". Furthermore, there are also three common causes between owner and contractor are: "price of construction materials increase very rapidly," shortages of skilled workers," and "political situation (revolution/ public strikes)".

In order to assess the delay causes by each party independently, the owners', consultants', contractors' and others' data were separated and analyzed individually. The top seven factors of delay were compared among different parties with their important index shows in Table 2. "Price of construction materials increase very rapidly" is identified as the top most influential cause of delay analysis by both the owner and consultant parties. This factor is also identified the number one ranked of delay cause by the overall result of analysis. Furthermore, the contractor results of analysis identified this cause as the second ranked, and third ranked factor of delay according to others' (Architects, sub-contractors etc). However, the most top important cause in the contractor's result is "shortages of skilled workers." It is ranked as the third in the owner's result and it is not listed within the seven important causes in the consultant's result. Thus the delay factor "shortage of skilled workers" is second ranked in the others party's results. Furthermore, the others party's result of analysis identified the first ranked factor of delay as poor site management and supervision by contractor". It is ranked as the third in the owner's result and it is not listed within the seven important causes in the consultant's result of analysis. Furthermore, it is fourth ranked listed in the overall result of analysis. "Incompetent/ immature subcontractors" is ranked as the second most important cause of delay in the consultant and the fourth ranked in

Table 5. Severity index and ranking

Factors	Owner		Consultant		Contractor		Others		Overall	
	SI	Rank	SI	Rank	SI	Rank	SI	Rank	SI	Rank
F1	0.575	13	0.511	22	0.500	6	0.667	11	0.555	14
F2	0.725	1	0.620	2	0.571	1	0.667	12	0.657	1
F3	0.488	26	0.457	32	0.500	7	0.472	35	0.470	32
F4	0.663	3	0.522	19	0.536	3	0.722	7	0.597	4
F5	0.538	21	0.554	12	0.429	16	0.639	16	0.547	17
F6	0.575	14	0.576	5	0.464	12	0.556	29	0.555	15
F7	0.588	10	0.565	8	0.500	8	0.639	17	0.572	8
F8	0.488	27	0.522	20	0.429	17	0.556	30	0.500	27
F9	0.550	20	0.533	17	0.536	4	0.583	24	0.542	19
F10	0.588	11	0.543	15	0.571	2	0.611	21	0.572	9
F11	0.488	28	0.554	13	0.429	18	0.639	18	0.530	20
F12	0.563	16	0.598	3	0.500	9	0.694	10	0.589	6
F13	0.500	23	0.500	25	0.393	24	0.667	13	0.513	25
F14	0.563	17	0.467	29	0.464	13	0.583	25	0.517	23
F15	0.513	22	0.565	9	0.321	30	0.556	31	0.513	26
F16	0.463	32	0.413	34	0.357	28	0.583	26	0.449	34
F17	0.463	33	0.359	35	0.500	10	0.500	34	0.428	35
F18	0.500	24	0.478	28	0.214	35	0.583	27	0.466	33
F19	0.688	2	0.457	33	0.393	25	0.722	8	0.568	11
F20	0.650	5	0.598	4	0.429	19	0.778	1	0.619	2
F21	0.600	7	0.576	6	0.429	20	0.778	2	0.597	5
F22	0.663	4	0.467	30	0.429	21	0.750	4	0.572	10
F23	0.600	8	0.641	1	0.393	26	0.778	3	0.614	3
F24	0.563	18	0.565	10	0.536	5	0.639	19	0.568	12
F25	0.500	25	0.576	7	0.321	31	0.611	22	0.525	21

F26	0.475	30	0.511	23	0.357	29	0.583	28	0.487	29
F27	0.438	35	0.522	21	0.393	27	0.556	32	0.483	31
F28	0.475	31	0.511	24	0.464	14	0.750	5	0.525	22
F29	0.450	34	0.543	16	0.321	32	0.750	6	0.517	24
F30	0.488	29	0.489	27	0.429	22	0.528	33	0.487	30
F31	0.563	19	0.467	31	0.321	33	0.611	23	0.500	28
F32	0.600	9	0.500	26	0.464	15	0.639	20	0.551	16
F33	0.588	12	0.565	11	0.500	11	0.722	9	0.589	7
F34	0.613	6	0.533	18	0.429	23	0.667	14	0.568	13
F35	0.575	15	0.554	14	0.286	34	0.667	15	0.547	18

the analysis of others party's result while it is not listed within the seven important causes in the owner's and contractor's results. The contractor and others parties identified the first ranked factor of delay as shortages of skilled workers, and poor site management and supervision by contractor respectively. The first ranked by the contractor, it is also the second ranked in the others party's results.

In order to appraise the reponisibility of group factors, data were seperated and analyzed groupwise. The influential top most cause from groups are: price of construction materials increase very rapidly, shortages of skilled workers, delay in running bill payments to contractor and financial difficulties of owner, poor/ deficient planning and estimate, poor site management and supervision by contractor, unrealistic project time estimation and imposed in contract, and political situation (revolution/ public strikes) according to Material, Manpower & equipment, Owner, Consultant, Contractor, Project, and External group respectively shows in Table 7. Their position in rank by overall analysis are: first, third, sixth, twenty ninth, fourth, twentieth, and second according to group.

Table 6. Importance Index and Ranking

Factors	**Owner**		**Consultant**		**Contractor**		**Others**		**Overall**	
	IMP. I	Rank	IMP.I	Rank	IMP.I	Rank	IMP.I	Rank	IMP.I	Rank
F1	0.302	16	0.294	13	0.304	3	0.444	12	0.318	9
F2	0.471	1	0.391	1	0.306	2	0.519	3	0.426	1
F3	0.225	31	0.184	34	0.196	16	0.197	34	0.197	34
F4	0.389	3	0.289	15	0.325	1	0.522	2	0.357	3
F5	0.302	15	0.307	9	0.214	12	0.408	14	0.308	12
F6	0.331	10	0.351	4	0.182	21	0.340	27	0.315	10
F7	0.294	18	0.307	10	0.232	9	0.373	19	0.301	16
F8	0.256	22	0.261	26	0.184	18	0.278	31	0.250	28
F9	0.316	13	0.255	28	0.287	4	0.340	25	0.290	17
F10	0.279	20	0.307	11	0.286	5	0.441	13	0.313	11
F11	0.232	28	0.331	8	0.199	14	0.408	15	0.290	18
F12	0.316	12	0.357	3	0.250	7	0.502	5	0.347	6
F13	0.238	27	0.288	16	0.196	17	0.389	17	0.274	22
F14	0.267	21	0.234	31	0.232	10	0.356	21	0.263	25
F15	0.250	23	0.283	19	0.092	34	0.309	30	0.245	29
F16	0.208	34	0.193	33	0.128	27	0.340	26	0.207	32
F17	0.225	32	0.164	35	0.214	13	0.264	33	0.201	33
F18	0.200	35	0.213	32	0.054	35	0.275	32	0.192	35
F19	0.361	7	0.258	27	0.154	23	0.461	9	0.308	13
F20	0.382	4	0.344	6	0.138	25	0.475	8	0.341	7
F21	0.390	2	0.351	5	0.138	26	0.540	1	0.357	4
F22	0.373	6	0.274	22	0.184	19	0.458	11	0.322	8
F23	0.360	8	0.369	2	0.140	24	0.519	4	0.351	5
F24	0.281	19	0.276	21	0.249	8	0.355	24	0.284	19
F25	0.244	24	0.282	20	0.103	30	0.356	22	0.254	26
F26	0.232	29	0.272	23	0.102	32	0.356	23	0.244	30
F27	0.230	30	0.284	17	0.126	28	0.370	20	0.254	27

F28	0.243	25	0.283	18	0.166	22	0.500	6	0.281	20
F29	0.242	26	0.307	12	0.103	31	0.479	7	0.276	21
F30	0.219	33	0.239	30	0.184	20	0.161	35	0.215	31
F31	0.323	11	0.249	29	0.115	29	0.340	28	0.265	24
F32	0.308	14	0.293	14	0.216	11	0.408	16	0.306	15
F33	0.382	5	0.332	7	0.268	6	0.461	10	0.359	2
F34	0.360	9	0.272	24	0.199	15	0.389	18	0.308	14
F35	0.302	17	0.271	25	0.102	33	0.333	29	0.266	23

Regarding the project size by estimated cost, the data were separated and analyzed according to small, medium, and large projects as shown in Fig. 8. In this case of analysis, there are three more common causes between medium and large projects are: "price of construction materials increase very rapidly," "difficulties in financing project by contractor," and "incompetent/ immature subcontractors". Here also mentioned that one most common cause affected the construction projects is "price of construction materials increase very rapidly". It is ranked in position first in the medium and large project, and fourth in the small projects.

Table 7. Ranking of group wise factors of delay

Group	Rank 1
Material	Price of construction materials increase very rapidly
Manpower and equipment	Shortages of skilled workers
Owner	Delay in running bill payments to contractor and financial difficulties of owner
Consultant	Poor/ deficient planning and estimate
Contractor	Poor site management and supervision by contractor
Project	Unrealistic project time estimation and imposed in contract
External	Political situation (revolution/ public strikes),

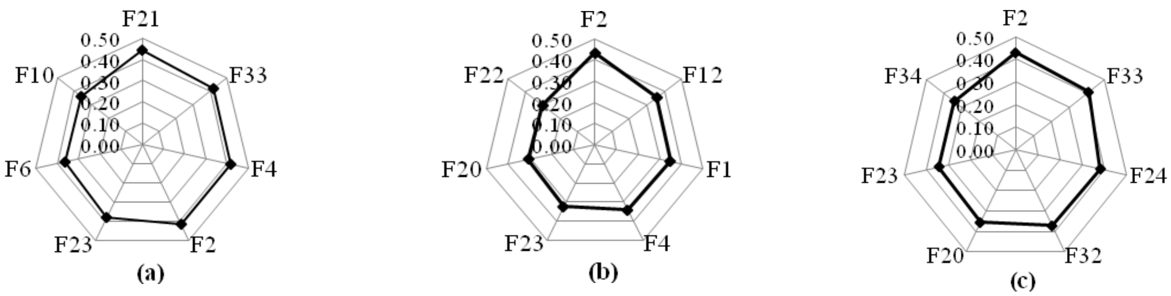

Fig 8. Spider mapping chart of important index of project size by total investment; considering (a) small project (b) medium project, (c) large project.

5. CONCLUSIONS

The objective of this study is to identify the main causes of delay that affect construction projects in Bangladesh. A comprehenssive literature review was conducted to identify the causes of delay stipulated in the literature. A compiled list of 35 factors of delay was selected and subjected to further quantitative investigation in a questionnaire survey to confirm the causes and identify the most important factors of project delay.

This study reveals that "price of construction materials increase very rapidly" is one of the most critical factors of construction delay. This finding of this research is indeed a clear contrast to the findings of Doloi et al. (2012) that the most important factor of delay is delay in material delivery by vendors. Political situation is certainly another important key factors affecting time performance of construction projects. It is the very common phenomenon or problem in the developing countries like Bangladesh. Other factors of delay are listed in the top seven: shortages of skilled workers, poor site management and supervision by contractor, incompetent/ immature subcontractors, delay in running bill payments to contractor and financial difficulties of owner, and difficulties in financing project by contractor. The results show near agreement between project parties in " price of construction materials increase very rapidly" being the top most factor of delay. All other causes witnessed disagreement and in some cases what can be considered "pinpointing" of responsibility of delay on other parties. For example, the contractor and others party identified "shortages of skilled workers" and " poor site management and supervision by contractor" as the first ranked factors of delay. However, the owner and consultant gave these factors of delay a lesser ranking. It is mentioned that the factor shortages of skilled workers is not enlisted in the top seven factors of delay by consultants.

Pearson's correlation of the responses of each party showed that there is an almost good agreement between parties. Among all pair of parties, four pair of parties (such as owner-consultant, owner-contractor, owner-others, and consultant-others) showed that a very good positive agreement in ranking these causes despite frequency, severity or importance index and another two pair of parties, consultant-contractor, and contractor-others showed positive relationships with lower level of agreement. Therefore, an analysis of the responsibilities of delay causes suggests that a joint effort based on teamwork is required to mitigate delays.

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REFERENCES

Abdul-Rahman, H., Berawi, M. A., Berawi, A. R., Mohamed, O., Othman, M., and Yahya, I. A. (2006) "Delay mitigation in the Malaysian construction industry." Journal of Construction Engineering Management, 132(2), 125–133.

Akogbe, R.T.M., Feng, X., and Zhou, J. (2013). "Importance and Ranking Evaluation of Delay Factors for Development Construction Projects in Benin." KSCE Journal of Civil Engineering, Vol. 17(6), 1213-1222

Al-Momani, A.H. (2000). "Construction Delay: A Quantitative Analysis." International Journal of Project Management, Vol. 18(1), 51–59.

Asaaf, S.A., and Al-Khalil, M. (1995). "Causes of Delay in Large Building Construction Projects." International Journal of Project Management, Vol. 11(2), 45-50.

Asaaf, S.A., and Al Hejji, S. (2006). "Causes of Delay in Large Construction Projects." International Journal of Project Management, Vol. 24, 349–357.

Baldwin, J. R., and Manthei, J. M. (1971). "Causes of Delay in the Construction Industry." Journal of Construction Division, ASCE, 97(2),177-187.

Chan, D.W., and Kumaraswamy, M.M. (1997). "A Comparative Study of Causes of Time Overruns in Hong Kong Construction Projects". International Journal of Project Management, Vol. 15(1), 55–63.

Doloi, H., Sawhney, A., Iyer, K.C., and Rentala, S. (2012). "Analyzing Factors Affecting Delays in Indian Construction Projects." International Journal of Project Management, Vol. 30, 479–489.

El-Razek M. E., Bassioni, H. A., and Mobarak, A. M. (2008). "Causes of Delay in Building Construction Projects in Egypt." Journal of Construction Engineering and Management, ASCE, Vol. 134(11), 831-841.

Ezeldin, A.S., and Abdel-Ghany, M. (2013). "Causes of Construction Delays for Engineering Projects in the Middle-East: An Egyptian Perspective." AEI, ASCE, 53-62.

Ibrahim, M., Bruland, A. and Dmaidi, N. (2012) "Causes of Delay in Road Construction Projects." Journal of Management in Engineering, ASCE, Vol. 24, 300-310.

Kaming, P., Olomolaiye, P., Holt,G., and Harris, F. (1997). "Factors Influencing Construction Time and Cost Overruns on High-Rise Projects in Indonesia." Construct Management and Economics, Vol. 15, 83–94.

Kamruzzaman, A.K.M., Said, I.B., Osman, O.B., and Hasmori, M.F. (2012). "Water Business - Public Private Partnership in Rural Bangladesh." 3rd International Conference on Business and Economic Research/ (3rd ICBER) Proceeding 12-13 March, Golden Flower Hotel, Bandung, Indonesia.

Le-Hoai, L., Lee, Y.D., and Lee, J.Y. (2008). "Delay and Cost Overruns in Vietnam Large Construction Projects: A Comparison with other Selected Countries." KSCE Journal of Civil Engineering, Vol. 12(6), 367-377

Love, P.E.D., Tse, R.Y.C., and Edwards, D.J. (2005) "Time-cost relationships in Australian building construction projects", *Journal of Construction Engineering and Management (ASCE)*, Vol. 131(2), 187-194.

Marzouk, M.M. and El-Rasas, T.I. (2013). "Analyzing Delay Causes in Egyptian Construction Projects." Journal of Advanced Research.

Shaon, MD.R.R. (2012). "Investigation of Construction Delay of Real Estate Companies in Dhaka City", Bachelor of Science thesis, Bangladesh University of Engineering and Technology, Dhaka, Bangladesh, Sweis, G., Sweis, R., Hammad, A.A., and Shboul, A. (2007). "Delays in Construction Projects: The case of Jordan." International Journal of Project Management, Article in Press.

EFFECTS OF CONSTRUCTION SEQUENCE ON A CONTINUOUS BRIDGE

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ABSTRACT

The construction of bridge superstructure is a highly complex process due to the interrelationship between the erection method used and the manifold internal and external effects related to loads and behavior and to environmental influence. At the time of construction of balanced cantilever bridge the moment in the bridge increases at each stage of addition of a new segment during construction. As the segments is added as cantilever form in balanced condition of both side of pier, the moment arise in the pier is negative. And it increases with the addition of each new segment. This increasing trend continues until the addition of last segment when it turns as a full cantilever bridge. After joining the last segment, developed positive moment and negative moment decreases significantly. By examining the results of the parametric study, it is seen that negative moment that developed before adding key blocks which is 30%-40% higher than the moment after full construction and deflection varies between 4 to13 cm. and these variation is nearly same for different height, width and span length. These are one of the key findings of this study. Before jointing key blocks the deflection is about 80%-100% higher than after full construction. The authors a bridge model is used to investigate this rate of change of moment with the addition of each segment. Also it is evaluated the rate of change of moment with different span length, superstructure width, and superstructure. The deflection at different stages of construction is also observed.

Keywords: Bridge, Construction Sequence, Balanced Cantilever Method, Increment of Moment

1. INTRODUCTION

Different means and methods exist in the construction of bridge superstructures. In planning and execution of the complex construction operations the effect of the chosen erection method need to be considered to achieve a safe and economical process. Failure of bridges during construction is importance of this issue.

At the time of construction of balanced cantilever bridge the moment in the bridge increases at each stage of addition of a new segment during construction. As the segments is added as cantilever form in balanced condition of both side of pier, the moment arise in the pier is negative. And it increases with the addition of each new segment. There increases negative moment until the last segment or the key block is added. When the key block is added, the bridge convert from cantilever to a continuous form and the negative moment on the pier decrease significantly and there also arise a positive moment which is also significant. So if a bridge is constructed using the balanced cantilever method and unless consideration of the construction stage moment, the bridge must be collapsed during construction stage. For this reason, the increment of moment during construction should be investigated.

A bridge model is used to investigate this rate of change of moment with the addition of each segment. Also to observe the rate of change of moment with span length, superstructure width, and superstructure height they are varied. The deflection at different stages of construction is also observed.

To perform this analysis, a finite element software, ANSYS is used. In the beam model, BEAM3 element is used and in the bridge, model SOLID45 element is used for the analysis. For the operation of segmental construction ANSYS Birth and Death feature is used.

From the study, the outcome is - in the construction of a continuous bridge in balanced cantilever method, (i) The moment increase almost parabolically during construction stages until the key block is added and the whole moment exist in negative form. (ii) The negative moment is maximum just before the addition of the key block. (iii) After the addition of the key block the negative moment decrease significantly on the pier and this final negative moment is about 60%- 70% of the maximum negative moment and there suddenly arise a positive moment at the mid – span which is about one – third of the maximum negative moment. (iv)The deflection also increase until the key block is added and after the addition of key block the deflection decrease significantly.

2. OBJECTIVES OF THE STUDY

There are several methods used in the construction of segmental bridge. One of them is balanced cantilever method. In this method the stages of construction is very important. Because, the moment and the deflection of the bridge change at every stage for the addition of a new segment. For this reason, the investigation of the rate of change of moment and deflection is highly required in different stage to consider the maximum moment and deflection for safe construction.

The main objective of this study is therefore to investigate the rate of change of moment and deflection at each stage of construction of a continuous bridge in the balanced cantilever method. Here, it is also investigated the rate of change of moment and deflection with the variation of span length, superstructure width and height.

3. METHODOLOGY OF THE STUDY

In the study of the balanced cantilever method of segmental bridge construction finite element software, ANSYS is used. Two models are built using ANSYS for this purpose. The first is a test model named beam model, and the second is the bridge model. In the beam model, the stage of construction is used to see the scope of this study using the software ANSYS. This operation is done using ANSYS Birth and Death feature. And it is observed that the objectives of the study will be fulfilled with this software.

Finally, the bridge model is used for the study. In this model the span length, superstructure width and height is varied to observe the effect of those parameters in the rate of change of moment and deflection.

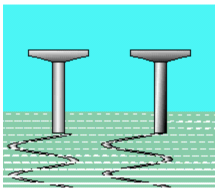
4. METHODS OF CONSTRUCTION OF SEGMENTAL BRIDGE

The different construction methods of segmental bridge are given below:

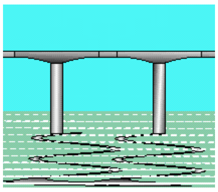
- Balanced cantilever method
- Span-by-span method
- Tarzan construction method
- Cantilever construction using temporary cable stays
- Cantilever construction method
- Incremental launching
- Lateral/transverse launching
- Longitudinal launching

4.1 Balanced Cantilever Construction Method

The balanced cantilever method was developed to minimize the false work required for in-situ construction. Temporary shoring is expensive especially in the case of high level bridges. Erection of false work crossing a river may be hazardous or even impossible. Over navigable waterways, trafficked roads or railways, false work is either not allowed or severely restricted. Cantilever construction eliminates such difficulties.



Using this method of construction the supporting pillars are constructed first. Then the sections which are to carry the deck of the bridge are built outwards, maintaining balance throughout.



To finish the process, the cantilevered sections are joined by link spans, which are dropped into place by crane.

5. NUMERICAL MODEL

5.1 Model l:

The test model of a beam is created using the finite element software ANSYS. In the finite element model, BEAM3 element is used. BEAM3 is used for the two - dimensional modeling of structures. This element is defined by two nodes having three degree of freedom at each node: translation in the node x, y, and rotation in z directions.

5.2 Mode 2:

The model of the proposed bridge is created using the finite element software ANSYS. In the finite element model, SOLID45 is used to represent the concrete. SOLID45 is used for the three - dimensional modeling of structures. This element is defined by eight nodes having three degree of freedom at each node: translation in the node x, y, and z directions.

6. MATERIAL PROPERTIES

Table 1: Concrete Properties (MODEL 2)

Young modules (Pa)	**Density** (Kg / m3)	**Poison ratio**
2.482e10	2400	0.2

7. THE PARAMETERS

The moment variation in the balanced cantilever method of a continuous bridge due to addition of each new segment is studied in several cases with variables span length, width and height.

Table 2: Parameter studied with certain Span, Width and different Height

SPAN (m)	**WIDTH (m)**	**HEIGHT(m)**
40	8	2
		3
		4
	10	2
		3
		4
	12	2
		3
		4
45	8	2
		3
		4
	10	2
		3
		4
	12	2
		3
		4
50	10	2

Table 3: Parameter studied with certain Width and Height and different Span

HEIGHT (m)	WIDTH (m)	SP AN (m)
2	10	40
		45
		50

8. MODEL WITH BEAM3 ELEMENT

For checking ANSYS death and feature at first analyze a simple beam of BEAM3 element of span 120ft. Here BIRTH element of construction at five stages sequentially considered. Figure 1 shows moment diagram of different stages .It is seen that at fourth stage before jointing key block the moment is about 80% higher at first pier and 20% higher at second pier.

Compare the value inscribed in the circular box of sequence 4 and 5 the negative moment at support reduced at final sequence and positive moment developed. From the figure it is also reveal that maximum negative moment developed at stage 4 i.e. previous stage of final stage.

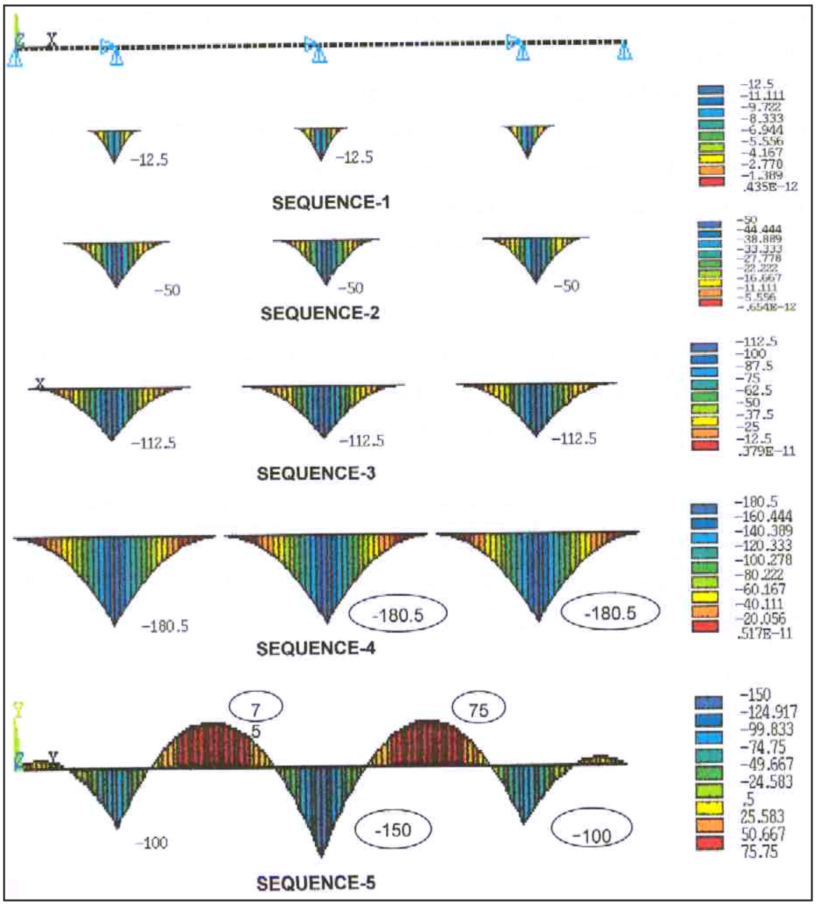


Figure 1: Sequential bending moment diagram of a continuous beam bridge

9. STAGES OF CONSTRUCTION

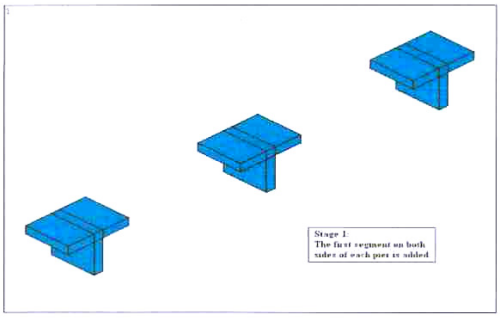


Figure 2: The first segment on both sides of each pier is added.

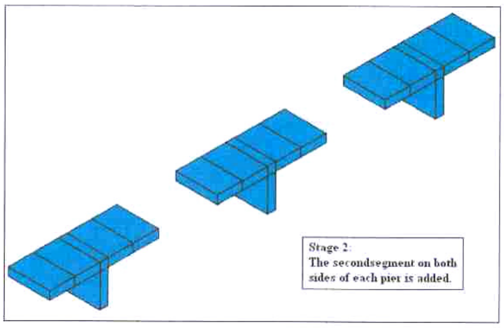


Figure 3: The second segment on both sides of each pier is added.

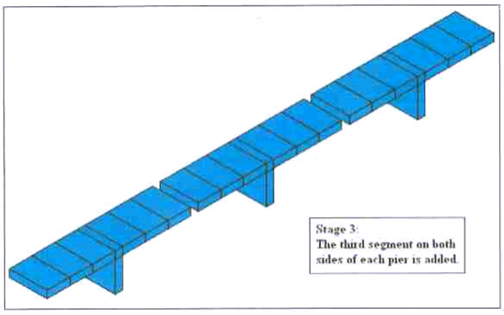


Figure 4: The third segment on both sides of each pier is added.

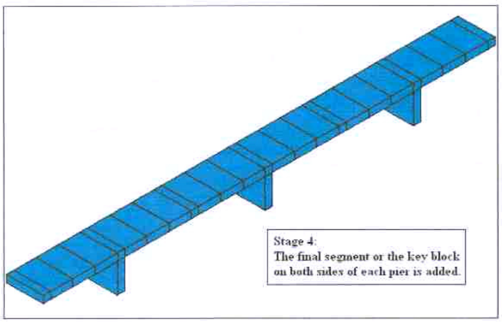


Figure 5: The forth segment on both sides of each pier is added.

10. BRIDGE MODEL WITH SOLID45 ELEMENT

In figure moment variation are shown in different stages (basically four stages of construction) of construction of model having Span length = 40 m, Superstructure Width = 8 m., Superstructure Height = 2 m. Fig represents comparison of moment diagram between before jointing key blocks and after full construction.

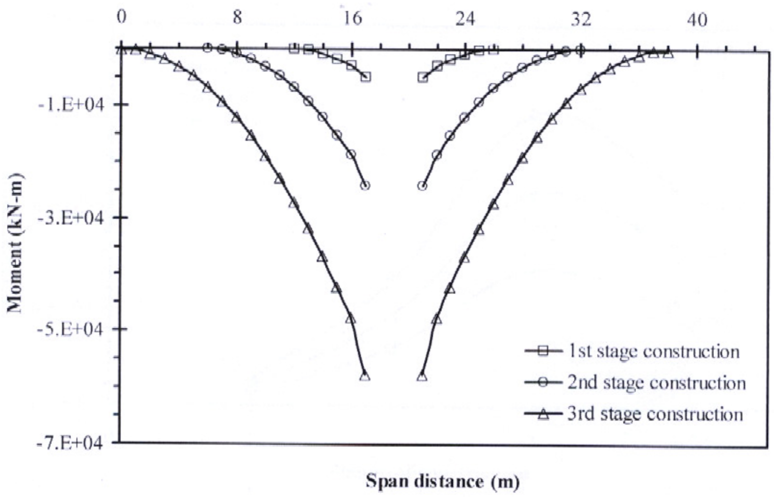


Figure 6: Moment diagram at different stages of construction with span=40 m, width=8 m, height=2m

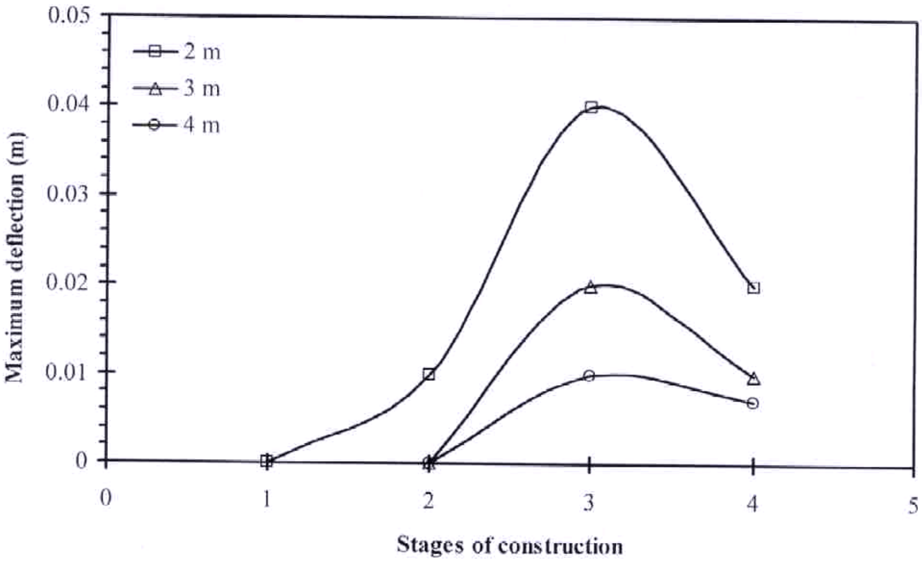


Figure 7: Comparison of Moment diagram before jointing key blocks and after full construction with span=40 m, width=8 m, height=2m

From figure 6 it is clear that before jointing key block elements (in 3rd stage) moment is about 40% higher than the moment after completing construction. Discontinuity of graph is due to presence of support. Moment of this section is omitted due to stress concentration.

In figure 8 some curve is drawn for maximum moment of different stages of construction with different superstructure height for Span length = 40m, Superstructure width = 8m. This curve shape is nonlinear in nature as the solution is nonlinear solution.

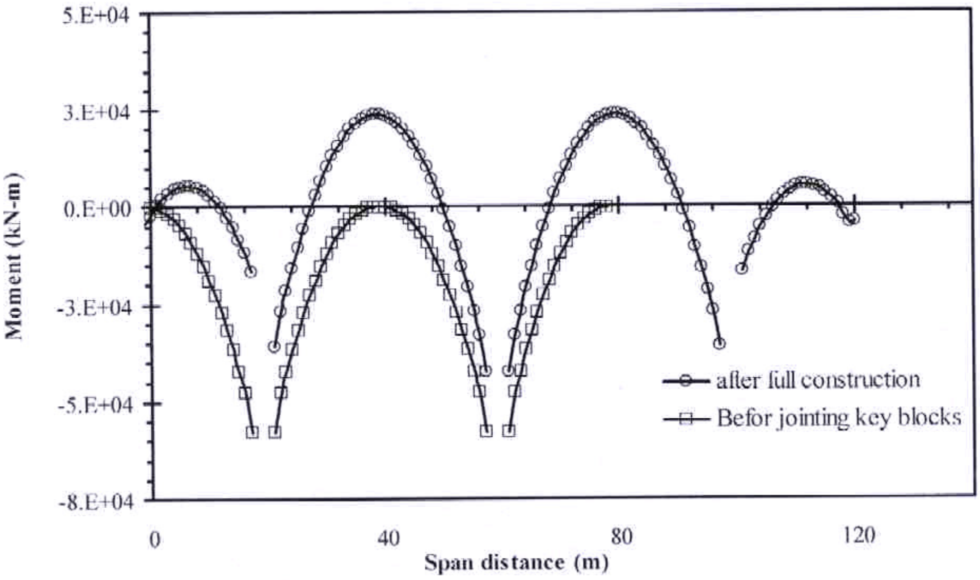


Figure 8: Maximum moment variation at different stages of construction of different super structure height with span=40 m, width=8 m

Another curve is plotted in figure 8 presenting the maximum deflection at various stages of construction with different superstructure height for Span length = 40m, Superstructure width = 8m. Here the curve of 2m height shows the maximum deflection at 3rd stage construction. Deflection is about 100% higher in 3rd stage construction than after full construction.

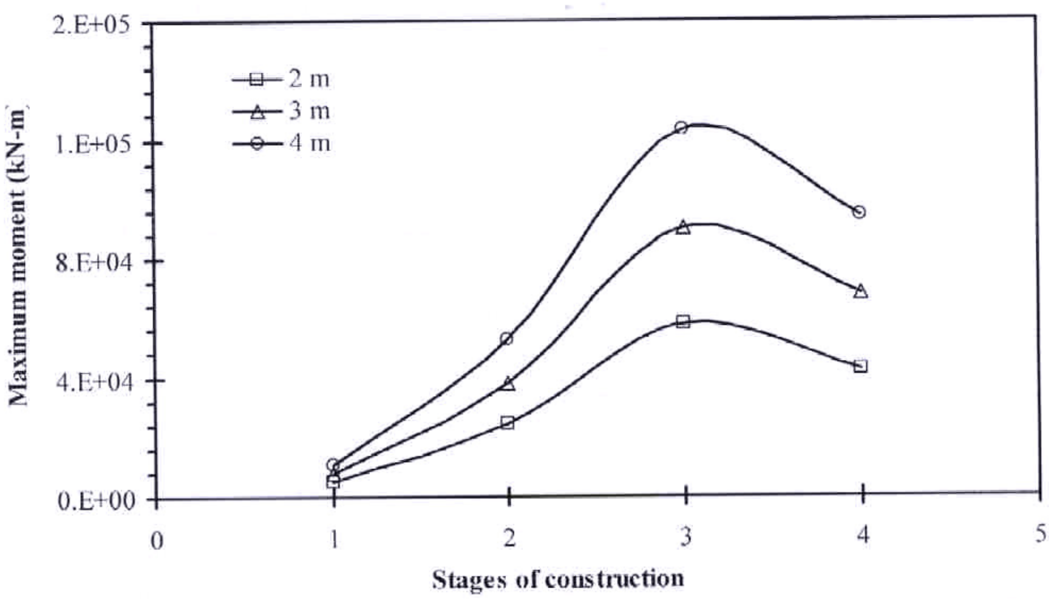


Fig 9: Maximum deflection variation at different stages of construction of different super structure height with span=40 m, width=8 m

In figure 10 Maximum moment variation are plotted at different stages of construction of different span length with width=10 m and height=2 m.

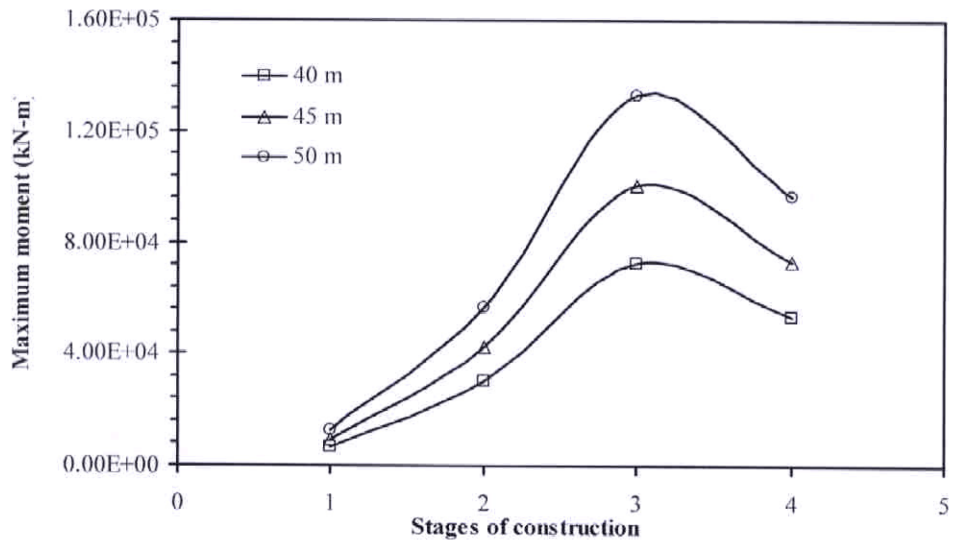


Fig 10: Maximum moment variation at different stages of construction of different span length with width=10 m, height=8 m

11. INCREMENT OF MOMENT FOR DIFFERENT WIDTH AND HEIGHT IN A CERTAIN SPAN

From the table 4, negative moment that developed before adding key blocks which is 30%-40% higher than the moment after full construction. From this it is also point out that percent increment is disproportion with the height i.e. minimum height maximum moment difference and vice versa for certain width and span length.

Table 4: Variation of moment for different width and height for a certain span length

SPAN (m)	WIDTH (m)	HEIGHT (m)	Moment Before jointing key blocks (KN-m)	Moment After full construction (KN-m)	% of difference before joining key block
40	8	2	58000	42500	37%
		3	90000	68000	33%
		4	123000	94100	31%
	10	2	72600	53200	37%
		3	112700	84800	33%
		4	153000	117600	30%
	12	2	87500	64000	37%
		3	135700	102000	33%
		4	184000	141000	30%

Table 5: Variation of moment for a certain width and height in different span length

WIDTH (m)	HEIGHT (m)	SPAN (m)	Moment Before jointing key blocks (KN-m)	Moment After full construction (KN-m)	% of difference before joining key block
10	2	40	72600	53200	37%
		45	100500	73300	37%
		50	133000	96600	38%

Variation of moment increases with the increasing of span length in a particular height and width (Table 5).

12. CONCLUSION

The study is undertaken to investigate the sequence of continuous bridge construction using Balanced Cantilever method. The emphasis of the parametric study is placed on the effect of moment and deflection in various stages of segmental construction with different span length, superstructure width and superstructure height. For simplicity we use solid rectangular superstructure (no hollow as used in segmental construction), which is erected segmentally. A pier consisting of transverse twin walls is advantageous as it provides stability for cantilevering but allows horizontal movement of the superstructure from thermal elongation through flexing of the wall panels.

The following conclusion may be drawn with respect to the cases studied in the parametric study. i) Due to self-weight stress pattern is linear throughout the cross section of superstructure. ii) Moment increases as the addition of new segment on each pier sequentially until jointing key blocks. iii) Before jointing key blocks moment is about 30%-50% higher than after full construction. iv) Moment increases with increasing model height, width, and span length. v) Deflection decreases with increasing superstructure height. vi) Deflection increases with increasing span length.

From the lesson of the present study, the recommendation for the future study may be summarized as follows. i) The parametric study could be done in a vast way, i.e., the span length, the superstructure width, and superstructure height of the bridge could be varied in a long range of data. ii) More realistic data of span length, the superstructure width, superstructure height of the bridge could be used. iii) The model could be analyzed with prestressing and realistic segment length. iv) For practical analysis different types of hollow (square, trapezoidal, and circular) section and realistic load (Equipment load, surcharge load, wind load, etc) could be used. v) A comparative economic study may be performed to compare construction cost between balanced cantilever and other construction methodology.

Finally it may be concluded that Continuous Bridge construction using the Balanced Cantilever methodology, much more considerations are needed during construction stage for overall economy and safety.

REFERENCES

Rowe, R. E. (1962). "Concrete Bridge Design", Books: London; John Wiley & Sons: New York. 1962.

O'Connor, C. (1971). "Design of bridge superstructures", New York; London, Wiley-Interscience. 1971.

Johnson Victor, D. (1991). "Essentials of Bridge Engineering", Oxford & IBH Publishing Co., New Delhi, Fourth Edition, 1991.

Podolny, Walter Jr. and Jean M. Muller. (1982). "Construction and Design of Prestressed Concrete Segmental Bridges".

AASHTO, "Standard Specification for HIGHWAY BRIDGE – 7th Edition. "

Chettoe, C. S. and H. C. Adams. (1948). "Reinforced Concrete Bridge Design", Chapman & Hall : London. 1948.

Gunnar Lucko, "Means and Methods Analysis of a Cast-In-Place Balanced Cantilever Segmental Bridge: The Wilson Creek Bridge Case Study", Thesis submitted to the Faculty of Virginia Polytechnic Institute and State University in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering.

STRENGTHENING TECHNIQUE OF REINFORCE CONCRETE COLUMN FOR ONE STOREY VERTICAL EXTENSION: A CASE STUDY OF CIVIL ENGINEERING BUILDING, KUET, BANGLADESH.

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ABSTRACT

Inadequate attention to Column during design and construction, adding new storey of Reinforced Concrete (RC) buildings in Bangladesh has raised questions about the performance level of Reinforced Column under future anticipated loads. RC jacketing and retrofitting by FRP to an existing column are common strengthening methods for column strengthening to increase the performance level of column to carry additional load. The aim of the study was concerned with the application of different strengthening methods to reinforced concrete column for one storey vertical extension when subjected to loads resulting from additional storey and selection of economical and sustainable strengthening method. As a case study, "Civil Engineering Building, KUET" was selected. To remodel of the existing building for one storey vertical extension, the survey works done included four main steps, making the column layout and identifying the RC Column, detecting the reinforcement bar for RC column, find out the capacity and additional load for RC column and selection of proper strengthening method in perspective of Bangladesh. After structural analysis, results showed that, the vertical load carrying capacity of the GF column is 124 Kips. But the column have to face 163 kips if one storey vertical extension will be done. So, the reinforced concrete columns need to be strengthened for vertical extension. After cost analysis, it was calculated that, for 1 Sq. meter syrface area of RC column,in total cost of FRP strengthening is approximately ten (10) times higher than the RC jacketing . So, RC jacketing of reinforced concrete column was selected by cost analysis and engineering view point analysis for the case study for adding additional storey.

Keywords: RC Column, RC jacketing, FRP Strengthening, Cost analysis, Engineering View Point Analysis.

1. INTRODUCTION

Buildingsmade in past time are mainly low-rise buildings. With the rapid development of construction, land becomes more and more scarce. As a result, construction of new building is quite expensive.

Strengthening technique of an existing structure plays an important role in mitigating the pressure of demand for more building sites, improving the living condition and accelerating the modernization progress. Therefore, now a day, the strengthening of an existing structure is considered to increase existing elements capacity to carry new loads or to resolve an existing deficiency. Over the years, engineers have used different methods and techniques to retrofit existing structures by providing external confining stresses. For the past few years, the concept of jacketing has been investigated to provide such forces. Externally applied jackets have been used as a reinforcement to contain concrete for different reasons. Engineers have used traditional materials such as wood, steel, and concrete to confine and improve the structural behavior of concrete members.

Section enlargement is one of the methods used in retrofitting concrete members. Enlargement is the placement of reinforced concrete jacket around the existing structural member to achieve the desired section properties and performance. The main disadvantages of such system are the increase in the column size obtained after the jacket is constructed and the need to construct a new formwork.

Interesting in the use of flexible fiber reinforced plastic (FRP) sheets for the external wrapping of concrete compressed members is today a very popular theme, especially as regards estimating the effectiveness of this reinforcing technique in increasing the strength and ductility of members in seismic areas.

The aim of the study was to determine the most effective and economical strengthening method for RC column among the mentioned methods in perspective of Bangladesh by cost analysis for one storey vertical extension.

2. METHODOLOGY

As a case study, "Civil Engineering Building, KUET" was selected. To remodel of the existing building for one storey vertical extension, the survey works done included four main steps, making the column layout and identifying the RC Column, detecting the reinforcement bar for RC column, find out the capacity and additional load for RC column and selection of proper strengthening method in perspective of Bangladesh. The reinforcement bar of column was identified from the previous design data which was designed in 1969 by chief engineer, East Pakistan. After the analysis of column to find out the capacity of column and additional load, RC jacketing and FRP strengthening was applied to the reinforced concrete column. Then the effectiveness of column strengthening method was investigated on the basis of cost analysis and engineering view point analysis.

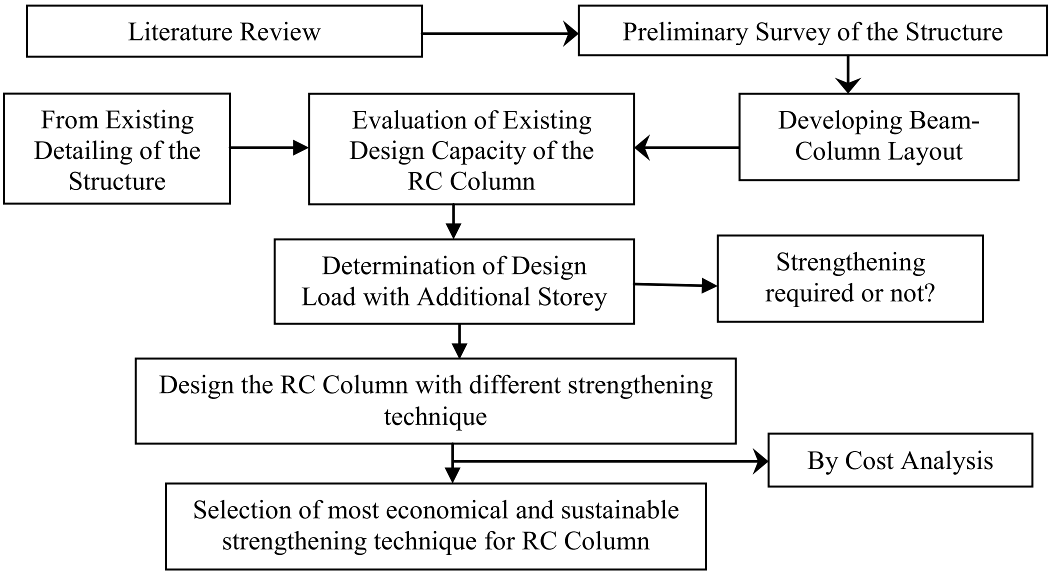

Figure 34: Existing Civil Engineering Building

Table 15: Material Strength Properties

Material Properties	Description
Yield Strength of used Reinforcement	40 ksi
Compressive Strength of used Concrete	3 ksi
Actual Compressive Strength of Concrete	2 ksi

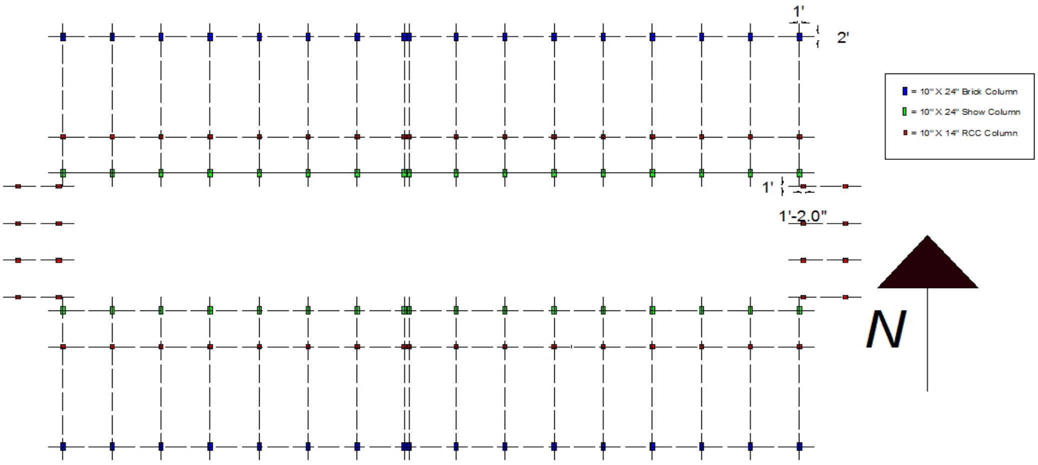

Figure 35: Beam-Column Layout

3. ANALYTICAL DETAILS

3.1 Capacity of RC Column

3.1.1 Ground floor column

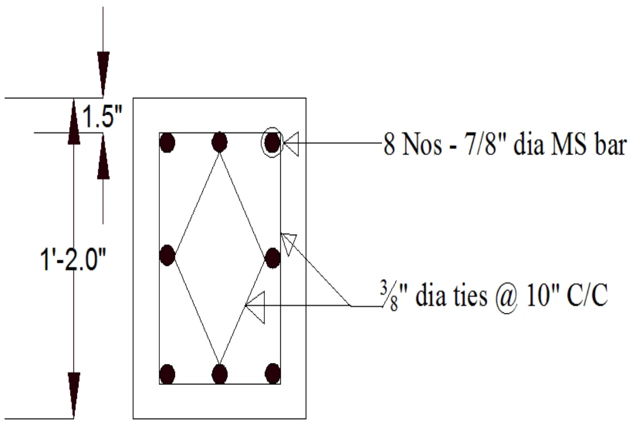

Figure 36: Ground Floor RC Column Section

Load Carrying Capacity

We Know, Load Carrying Capacity, $P = 0.85[A_g(0.25f'_c+\rho_g f_s)]$

Where, $A_g \longrightarrow$ Gross Cross-Sectional Area of Column = 10 * 14 = 140 inch^2

Area of Steel, $A_{st} = 8 * \pi/4 * (7/8)^2 = 4.81$ inch^2

Reinforcement ratio, $\rho_g = A_{st}/ A_g = 4.81/140 = 0.03436$
f'_c=2000 psi; f_y=40000 psi (from the detailing of existing structure)

Now, Load Carrying Capacity, P = 0.85[140 * (0.20 * 2000 + 0.03436 * 16000)
P =124.921 Kips ≅ 124 Kips

Moment Carrying Capacity

Let, Compression Govern –

$f_a/F_a + f_b/F_b \leq 1$ ……………… (1) [For Economical]

Where, $fa = P/A_g$= 124.921/144 = 0.8923; $f_b = M/S_{ut}$
$F_a = 0.34 (1+ m \rho_g) * f'_c$; Where, $m = f_y/(0.85 * f'_c) = 40/(0.85*2) = 23.529$
So, F_a = 0.34 (1 + 23.529 * 0.03436) * 2 = 1.23
$F_b = 0.45 * f'_c$ = 0.45 * 2 = 0.9 ksi
$S_{ut} = 1/c [b * h^3 / 12 + 2(2n – 1) * A_{st}/2 * (h/2 – d)^2]$
=> $S_{ut} = 1/7 [10 * 14^3 / 12 + 2(2*11.376 – 1) * 4.81//2 * (14/2 – 1.5)^2]$
=> S_{ut} = 778.805

Now, (1) => 0.8923 / 1.23 + (M / 778.805) / 0.9 = 1
So, Moment Carrying Capacity, M =192.441 Kips - inch. ≅ 192 Kips

Now, Developed Ecentricity, $e_{dev} = M_{cal}/P_{cal}$ = 192.441 / 124.921 = 1.541 inch
Balanced Ecentricity, $e_b = (0.67 * \rho_g * m + 0.17) * d$
=> e_b = (0.67 * 0.03436 * 23.529 + 0.17) * 12.5
=> e_b = 8.896 > e_{dev} = 1.541 inch (OK)

So, Load Carrying Capacity of Ground Floor Column, P = 124 Kips &
Moment Carrying Capacity of Ground Floor Column, M = 192 Kips - inch

3.1.2 First floor RC column

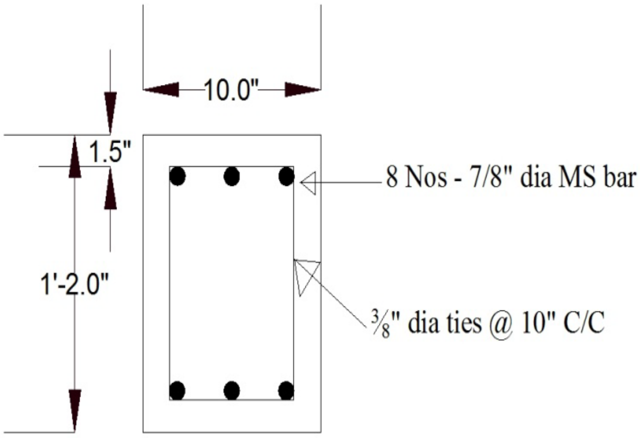

Figure 37: First Floor RC Column Section

Load Carrying Capacity

We Know, Load Carrying Capacity, $P = 0.85[A_g (0.25f'_c+\rho_g f_s)]$
Where, $A_g \longrightarrow$ Gross Cross-Sectional Area of Column = 10 * 14 = 140 inch^2
Area of Steel, $A_{st} = 6 * \pi/4 * (7/8)^2 = 3.608$ inch^2
Reinforcement ratio, $\rho_g = A_{st}/ A_g = 3.608/140 = 0.0258$
f'_c=2000 psi; f_y=40000 psi (From the detailing of existing structure)

Now, Load Carrying Capacity, P = 0.85[140 * (0.20 * 2000 + 0.0258* 16000)
P =96.668 Kips ≅ 96 Kips

Moment Carrying Capacity

Let, Compression Govern –

$f_a/F_a + f_b/F_b \leq 1$ ……………… (1) [For Economical]

Where, $f_a = P/A_g = 96.668/140 = 0.6905$; $f_b = M/S_{ut}$

$F_a = 0.34\ (1 + m\ \rho_g) * f'_c$; Where, $m = f_y/(0.85 * f'_c) = 40/(0.85*2) = 23.529$

So, $F_a = 0.34\ (1 + 23.529 * 0.0258) * 2 = 1.093$

$F_b = 0.45 * f'_c = 0.45 * 2 = 0.9$ ksi

$S_{ut} = 1/c\ [b * h^3 / 12 + 2(2n - 1) * A_{st}/2 * (h/2 - d)^2]$

=> $S_{ut} = 1/7\ [10 * 14^3 / 12 + 2(2*11.376 - 1) * 3.608//2 * (14/2 - 1.5)^2]$

=> $S_{ut} = 584.185$

Now, (1) => $0.6905 / 1.093 + (M / 584.185) / 0.9 = 1$

So, Moment Carrying Capacity, M =192.441 Kips - inch.

Now, Developed Ecentricity, $e_{dev} = M_{cal}/P_{cal} = 193.615 / 96.668 = 2.003$ inch

Balanced Ecentricity, $e_b = (0.67 * \rho_g * m + 0.17) * d$

=> $e_b = (0.67 * 0.0258 * 23.529 + 0.17) * 12.5$

=> $e_b = 7.21 > e_{dev} = 2.003$ inch (OK)

So, Load Carrying Capacity of First Floor Column, P = 96 Kips &
Moment Carrying Capacity of First Floor Column, M = 192 Kips - inch

3.2 Design Load on Column

Design Load on Slab

Let,

Thickness of the new Slab = 5 inch
Live Load, LL = 40 psf
Floor Finish, FF = 20 psf
Partition Wall, PW = 30 psf

Now, Self-Weight of the Slab = 5 / 12 *150 = 62.5 psf

Total Load on Slab = (62.5 + 40 + 20 + 30) =152.5 psf

Design Load on Beam

Load from Slab on each Beam = 152.5 psf * 12.0833 / 2 = 915 lb/ft
Self-Weight of Beam = 150 * 29 * 10 /144 = 302.083 lb/ft
Dead Load from Partition Wall = 450 lb/ft
Total Load on Beam = 915 + 302.083 + 450 = 1667.083 lb/ft = 1.667 K/ft

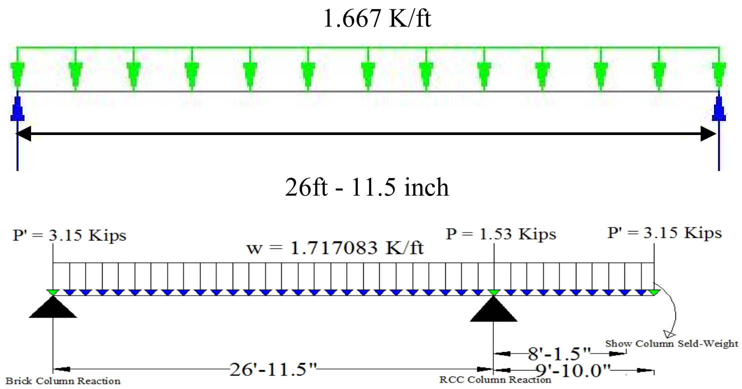

Figure 38: Distributed Uniform Load on Beam

Design Load on Column

Third Floor Column

Load on RCC Column from Beam = 1.667 * [{(26 + 11.5/12) / 2} + {8 + (1. / 12)}]
= 35.94 Kips

Self-Weight of Column (10 inchX14 inch) = 10*14/144(ft2)*Height of Column (ft.)*150 lb/ft3
= 10 * 14 / 144 * 10.5 * 150 lbs
= 1531.25 lb = 1.53 Kips
Self-Weight of (12 inch X 24 inch) Column = (12 * 24 / 144) * 10.5 * 150 lbs
= 3150 lbs = 3.15 Kips

Total Load on RCC Column (3rd Floor) = 35.94 + 1.53 + 3.15 = 40.62 Kips

Second Floor Column

Load on RCC Column from Beam = 35.94 Kips
Load from 3rd Floor Column = 40.62 Kips
Total Load on RCC Column (2nd Floor) = 1.53+35.94+40.62+3.15 = 81.24 Kips

First Floor Column

Load on RCC Column from Beam = 35.94 Kips
Load from 2nd Floor Column = 81.24 Kips
Total Load on RCC Column (1st Floor) = 1.53+35.94+81.24+3.15 = 121.86 Kips

But,
Load Carrying Capacity of 1st Floor Column = 96 Kips > Design Load (=121.86 Kips)

So, The First Floor Column Need To Be Strengthened

Additional Load on First Floor Column from Super Structure = 121.86 – 96
= 25.86 Kips ≅ 26 Kips

Additional Load on First Floor Column is 26 Kips

Ground Floor Column

Load on RCC Column from Beam = 35.94 Kips
Load from 1st Floor Column = 121.86 Kips

Assume, 2.5 ft from Floor level to Ground level.
So, additional self-weight = 10 * 14 /144 ft2 * 2.5 ft * 150 lb / ft3
= 364.583 lbs = 0.365 Kips

Total Load on RCC Column (Ground Floor) = 1.53+3.15+35.94+121.86+0.365
=162.845 Kips ≅ 163 Kips
But,
Load Carrying Capacity of GF Column = 124 Kips > Design Load (= 163 Kips)

So, The Ground Floor Column Need To Be Strengthened

Additional Load on Ground Floor Column from Super Structure = (163– 124) Kips
= 39 Kips

Additional Load on Ground Floor Column is 39 Kips

3.3 Strengthening of RCC Column

3.3.1 Strengthening of RCC column by RC jacketing

Procedure for RC Jacketing of Column

Additional Load on Ground Floor Column = 44 Kips
We Know, Load Carrying Capacity, P = 0.85[Ag (0.25f'c+ρgfy)]

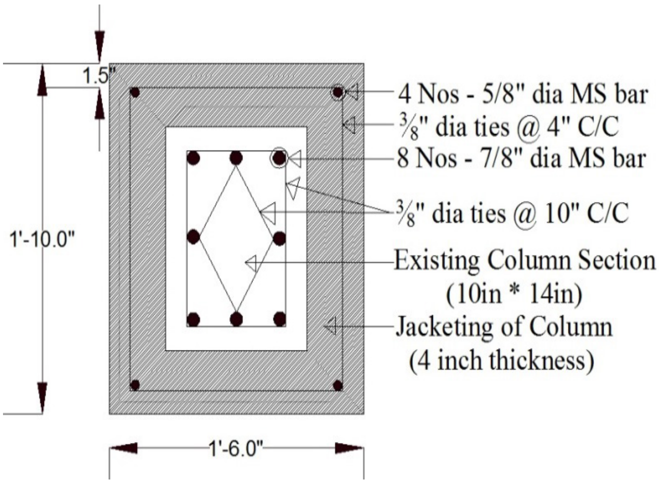

Figure 39: RC jacketing of ground floor RCC column section

By ACI code Requirement,

Minimum enlargement of column side (each) = 4 inch
Minimum reinforcement used (main bar) = 5/8 inch
Minimum No. of Reinforcement = 4 Nos.
Minimum diameter of tie Rod = 3/8 inch
Vertical spacing of tie Rod = 4 inch

So, new column section will be = 14 inch * 18 inch

Where,
$A_g \longrightarrow$ Total Gross Cross-Sectional Area of Column = (18*22) = 396 inch^2
$A_{g1} \longrightarrow$ Existing Gross Cross-Sectional Area of Column = 10*14 = 140 inch^2
$A_{g2} \longrightarrow$ Additional Gross Cross-Sectional Area of Column = 396-140 = 256 inch^2

Existing Area of Steel, $A_{st1} = 8 * \pi/4 * (7/8)^2 = 4.81$ inch^2
Additional Area of Steel, $A_{st2} = 4 * \pi/4 * (5/8)^2 = 1.227$ inch^2
Total Area of Steel, $A_{st} = A_{st1} + A_{st2} = 4.81 + 1.227 = 6.037$ inch^2

Reinforcement ratio, $\rho_g = A_{st}/ A_g = 1.227/252 = 0.005$
Use, $f'_c = 3000$ psi and $f_y = 50000$ psi (50 grade steel)

Now, $P = 0.85[A_g (0.25f'_c+\rho_g f_y)]$

$=>P = 0.85[140*(0.25*2000+0.03436*16000)] + 0.85[252*(0.25*3000+0.005*20000)]$

=> P = 306.99 Kips > Design Load = 163 Kips

Cost Analysis of Column after RC Jacketing

Total Height of Column = Below PL + Ground Floor Height + Ground Floor Height
+ Second Floor Height + Extended Height

= (4.5 + 10 + 10 + 10.5 + 2.5) = 37.5 ft

Determination of Required Reinforcement

Now, Volume of additional main bar = 4 * π/4 * $(5/8)^2$ * 37.5 *12 =552.23 $inch^3$

Weight of 5/8 inch diameter main bar = 490 lb/ft^3 * (552.23/1728) ft = 156.59 lb
= 71.02 kg ≅ 72 kg

No. of tie used = 37.5 *12 / 4 = 113 Nos.

Volume of additional tie bar = [{(22-1.5) + (18-1.5)}*2] * 113 * π/4 * $(3/8)^2$
= 923.55 $inch^3$

Weight of 3/8 inch diameter main bar = 490 lb/ft^3 * (923.55/1728) ft = 261.89 lb
= 118.79 kg ≅ 119 kg

Total weight of reinforcement = 72 kg + 119 kg = 191 kg (for one column)

Determination of Required Cement, Sand and Khoa

Wet volume of Additional concrete = 256 in^2 *(35 * 12) = 107520 in^3 = 62.22 ft^3
Dry volume of Additional concrete = 62.22 * 1.5 ft^3 = 93.33 ft^3 ≅ 94 ft^3
Reinforcement = (552.23 + 923.55) / 1728 = 0.854 ft^3 ≅1 ft^3
Net Volume of Cement Concrete = 94 – 1 = 93 ft^3
Use mixing ratio = 1: 1.5:3

So, Volume of Cement = 93 * 1 / 5.5 = 16.9 ≅ 17 ft^3
Volume of Sand = 93 * 1.5 / 5.5 = 25.4 ft^3 ≅ 26 ft^3
Volume of Brick Khoa = 93 * 3 / 5.5 = 50.73 ft^3 ≅ 51 ft^3

Weight of Cement = 90 lb/ft^3 * 17 ft^3 = 1530 lb = 694 kg ≅ 14 Bags
Weight of Sand = 120 lb/ft^t * 26 ft^3 = 3120 lb = 1415.21 kg ≅ 1416 kg

For 100 ft^3 Brick Khoa require brick = 850 Nos.
So, for 51 ft^3 brick Khoa required brick = 433.5 Nos. ≅ 434 Nos.
≅ 450 Nos. (Considering loss)

Machine cost for brick crusher = TK. 500 / 1000 Nos.

For Plastering and White Washing Work,

Surface Area = 35 * 12 * 2 * (22+18) = 33600 in^3 = 19.44 ft^3 ≅ 20 ft^3

Table 16: Cost analysis for RC jacketing of column

Items	**Required**	**Unit Cost**	**Cost (TK)**
Reinforcement	191 Kg	TK. 55000 / 1000 Kg (50 Grade)	10505
Cement	14 Bags	TK. 500 / per bag	7000
Sand	26 cft	TK. 35 / cft (Sylhet Sand)	910
Brick	450 Nos.	TK. 7500 / 1000 Nos.	3375
Brick Crusher Machine	450 Nos.	TK. 500 / 1000 Nos.	225
Plastering Work	20 ft^2	TK. 110 / ft^2	2200
White Washing	20 ft^2	TK. 7.5 / ft^2	150
Total			24365
Contigency	5 %		1219
Total			25584

Total cost in TK of RC Jacketing for 140 in^2 of RCC Column = 25584
Total cost in TK for RC Jacketing per m^2 of surface area of RC Column = 25584 * 1 / 0.090322
= 283253.2495 ≅ 300000 (Including lump-sum)

So, Total Cost for Strengthening by RC Jacketing of per square meter of Surface Area of RCC Column for one Storey Vertical Extension is TK. 300000

3.3.2 Strengthening of RC column by FRP

An E-glass / epoxy FRP complete wrap is selected to retrofit the column. The properties of the FRP system are given in Table 3. The design calculations to arrive at the number of complete wraps required follow:

Table 17: Manufacturer's reported FRP system properties

Thickness per ply, t_f	0.051 in.
Guaranteed ultimate tensile strength, f^*_{fu}	80,000 psi
Guaranteed rupture strain, e^*_{fu}	0.020 in./in.
Modulus of elasticity, E_f	4,000,000 psi

Procedure for Strengthening of Column by FRP

STEP-1: COMPUTE THE DESIGN MATERIAL PROPERTIES

The column is located in an exterior environment and a Glass Fiber Reinforced Polymer (GFRP) material will be used. Therefore, an environmental-reduction factor of 0.65 is suggested.

$$f_{fu} = C_E\, f_{fu}^* = (0.65) * (80 \text{ ksi}) = 52 \text{ ksi}$$
$$\varepsilon_{fu} = C_E\, \varepsilon_{fu}^* = (0.65) * (0.020) = 0.013$$

STEP-2: CALCULATE THE EFFECTIVE STRAIN LEVEL IN THE FRP SHEAR REINFORCEMENT

The effective strain in a complete FRP wrap can be determined from the following equation

$$\varepsilon_{fe} = 0.004 \le 0.75\varepsilon_{fu} = 0.004 \le 0.75(0.013) = 0.010$$

$\therefore$ Use an effective strain of $\varepsilon fe = 0.004$

STEP-3: DETERMINE THE AREA OF FRP REINFORCED REQUIRED

The required shear contribution of the FRP reinforcement can be computed based on the increase in strength needed, the strength-reduction factor for shear, and a partial-reduction factor of 0.95 for completely wrapped sections in shear.

$$V_{f,\, reqd} = \Delta\, V_u \,/\, \phi(\psi) = 39 / (0.85 * 0.95) = 48.297 \text{ Kips}$$

The required area of FRP can be determined by following equation. The required area is left in terms of the spacing.

$$A_{fv,\, reqd} = (V_{f,\, reqd} * s_f) / [\varepsilon_{fe}\, E_{fe}\, (\sin\alpha + \cos\alpha)\, d_f]$$
$$A_{f,\, reqd} = (48.297 \text{ kips}) * s_f / (0.004) * (4000 \text{ ksi}) * (1) * (14 \text{ in}) = 0.2156\, s_f$$

STEP-4: DETERMINE THE NUMBER OF PLIES AND STRIP WIDTH AND SPACING

The number of plies can be determined in terms of the strip width and spacing as follows:

$$n = A_{f,\, reqd} / (2 * t_f * w_f) = (0.2156 * s_f) / (2 * 0.051 \text{ in} * w_f) = 2.114\, s_f / w_f$$

$\therefore$ Use three plies (n = 3) continuously along the height of the column ($sf = wf$).

Cost Analysis of FRP Strengthening

To cover 1 square meter 0.5 Litre Epoxy Resin is needed.
Total Surface Area of the Column (Ground Floor to 2^{nd} Floor) = 37.5 *12 * {2 * (10 +14)}

= 21600 inch^2 = 14 m^2

So, total Epoxy Resin is needed = 0.5 L/m^2 * 14 m^2 = 7 L

Cost of FRP strengthening per column is given below:

Table 18: Cost analysis for RC jacketing of column

ITEMS	Unit Cost	Unit	Cost (TK)
Epoxy Resin	TK. 3000 / L	7 L * 3 (No. of ply) = 21 L	63000
Primer	TK. 540 / L	7 L	3780
FRP sheet	TK. 8500 / Sq. Meter	14 Sq. Meter	119000
Installation Cost	TK. 3000 / Sq. Meter	14 Sq. Meter	42000
Total			**227780**

Here there is no need of using putty, as the column hasn't big hole and crack.

Total cost in TK of RC Jacketing for 140 in^2 of RCC Column = 227780
So, Total cost in TK for RC Jacketing for 144 in^2 or per ft^2 RC Column = 234288
Total cost in TK for RC Jacketing per square meter of RC column = 234288 * 1 / 0.090322
= 2593919.533 ≅ 3000000 (Including lump-sum)

Per Ssquare Meter Total Cost for FRP strengthening of RC Column for one Storey Vertical Extension is TK. 3000000

4. RESULTS AND DISCUSSION

From Table 1, it is seen that, ground floor column and first floor column are merely carry the load from existing structure. After adding additional storey the design load exceeds the capacity of the column and strengthening must needed.

The capacity and analysis of the RC column are given to the Table 1:

Table 19. Analysis of RC Column (10 inch * 14 inch)

Structural Element	Capacity (Kips)	Load (Kips) from		Additional Load (Rounding) (Kips)	Strengthening Required?
		Existing Structure	After adding storey		
Ground Floor Column	125	121	163	39	Yes
First Floor Column	96	81	122	26	Yes

Total cost of strengthening method per square meter of surface area for each column to carry additional load is given to the Table 2:

Table 20: Cost Required for Different Strengthening Method

Strengthening Method	Total Cost in TK. for (140 in^2)
RC Jacketing	300000
FRP	3000000

Table 2 shows that, total cost per square meter (surface area) for FRP strengthening is approximately ten times higher than the RC jacketing. For FRP strengthening, installation cost and the materials needed for are quite high. Due to this the cost needed per column is much greater than RC jacketing.

5. CONCLUSIONS

In developing countries the labor cost is low but the material is relatively expensive, so as the cost of FRP. For FRP strengthening, skilled labor also needed which is a limitation for this type of strengthening. That's why, total cost required for FRP strengthening was found approximately ten (10) times higher than the RC jacketing. Due to this limitation and high cost, the FRP strengthening technique is quite impracticable in Bangladesh, though it shows greater output result and higher protection to corrosion and provide minimum disturbance to the existing structure.

Therefore, based on the above discussion it might be concluded column jacketing are the most economic strengthening method for reinforced concrete column for adding additional storey to the existing civil engineering building.

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REFERENCES

ACI Committee 440, "Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structure", 28th May, 2002, pp. 101-103.

ACI International, BRE, Concrete Society and ICRI, 2003. "Guide to the Maintenance, Repair and Monitoring of Reinforced Concrete Structures, Concrete Repair Manual", 2nd Edition, May. ISBN: 0-87031-105-0, 1: pp. 691-736.

Aziz M. A., A Textbook of Estimating and Costing, Ch. 2, pp. 21-35(1967).

Detailing of the Civil Engineering Building (1969), Chief Engineer, East Pakistan.

Proceedings of the 2nd International Conference on Civil Engineering for Sustainable Development (ICCESD-2014), 14~16 February 2014, KUET, Khulna, Bangladesh, ISBN: 978-984-33-6373-2 (CD-ROM)

INVESTIGATION OF DIRECT TENSION CAPACITY OF STEEL FIBER REINFORCED CONCRETE (SFRC): FINITE ELEMENT (FE) ANALYSES OF EXPERIMENTAL OUTCOMES

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ABSTRACT

Concrete shows a low tensile strength in comparison to the compressive strength, which grows only less proportionally with increasing compressive strength; at the same time, the brittleness increases. Steel fibers as reinforcement improves the tensile strength and ductility of concrete and reduce the possibility of catastrophic brittle failure during earthquake or other deadly loads although conventional reinforcement at the tension zone is compulsory. To this end, this research provides the evaluation of the capacity enhancement of steel fiber reinforced concrete (SFRC) and also Finite Element (FE) analyses to predict the direct tension capacity. To understand the tensile behaviour of SFRC, dog-bone specimens are casted and tested in direct tension. FE models of dog bone specimens are analyzed and validated with the experimental outcomes to predict the direct tensile capacity of SFRC. ANSYS 10.0 finite element platform is used to model SFRC. The governing parameters, such as, Poisson's ratio, stress strain relationship, tensile strength, modulus of elasticity, shear transfer coefficients for open and close crack are extensively analyzed to get accurate response from FE models. The construction industry of Bangladesh yet not started construction of structures made of SFRC due to lack of reliable experimental data and FE models. This paper provides FE model of SFRC validated with the experimental outcomes which will be helpful to predict the tensile behaviour of structures made of SFRC.

Keywords: Steel fiber reinforced concrete (SFRC), ANSYS, Finite Element (FE) modelling and analysis, direct tension of concrete, dog-bone specimen.

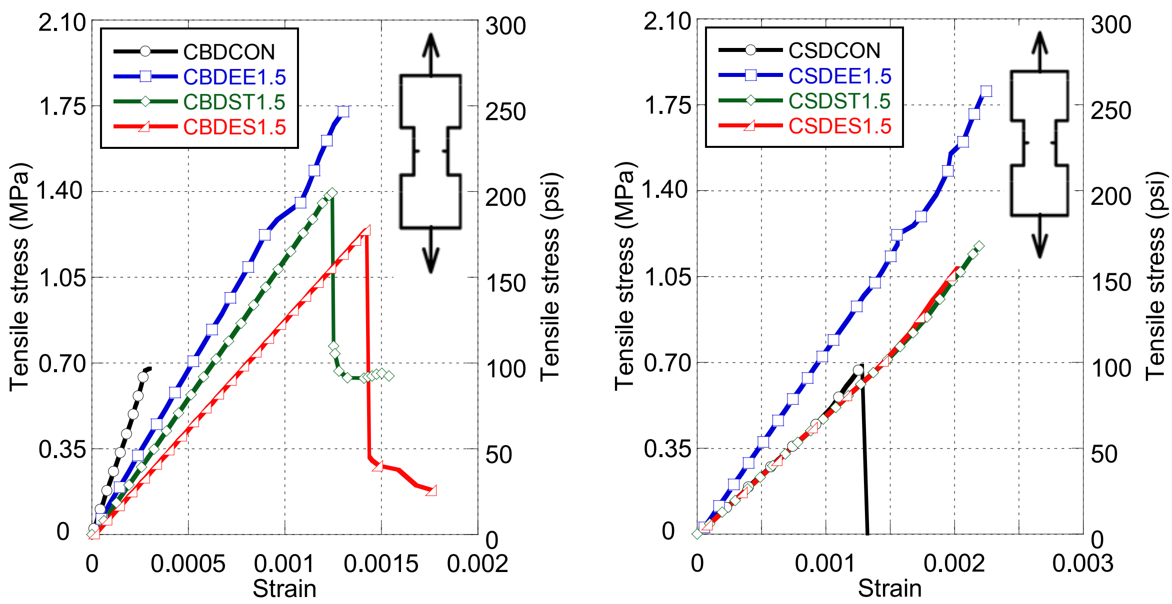
1. INTRODUCTION

Concrete shows a low tensile strength in comparison to the compressive strength, which grows only less proportionally with increasing compressive strength; at the same time, the brittleness increases. Therefore, fibers are added to improve ductility and to increase the tensile strength. For many applications, however, conventional reinforcement in the tensile zone is still necessary. Fibers are increasingly being used in concrete structures to compensate for concrete's weak and brittle tensile behaviour relative to its compression response. One of the most beneficial aspects of the use of fibers in concrete structures is that non-brittle behaviour after concrete cracking can be achieved with fibers. The tensile stress sustainable in concrete rapidly decreases immediately after cracking. In fiber reinforced concrete (FRC), on the other hand, fibers crossing the crack interfaces significantly contribute to the load-carrying mechanism so that considerable tensile stress, being the sum of the tensile resistance provided by fibers and tension softening of the concrete matrix, respectively, can be achieved even with large crack widths (Lee et al 2011). Therefore, the enhanced tensile stress behaviour attainable with fibers should be realistically evaluated to accurately predict the post-cracking response of FRC. The bond resistance of reinforcing bars embedded in concrete depends primarily on frictional resistance and mechanical interlock. The chemical adhesion bond, if any, fails at very small slips. Frictional bond provides initial resistance against loading and further loading mobilizes the mechanical interlock between the concrete and bar ribs. Mechanical interlock leads to inclined bearing forces, which in turn lead to transverse tensile stresses and internal inclined splitting (bond) cracks along reinforcing bars (Chao et al 2009). For many years, ACI 544.4R-88 has been working towards the development of standardized testing techniques as applied to fiber reinforced concrete. The committee suggested that the work is not finished and a continuous research effort is needed to improve testing and reporting methods for SFRC. To this end, this research modelled direct tensile strength of plain concrete and steel fiber reinforced concrete (SFRC) in Finite Element platform and are evaluated based on experimental investigation.

The ANSYS 10.0Finite Element Analysis (FEA) software package is used to analyze the direct tension specimens and introduce a good concrete model for Steel Fiber Reinforced Concrete (SFRC) as well as plain concrete made of brick and stone aggregate. Two different Poison's ratios for brick and stone concrete are selected by comparing FE output with the stress-strain behaviour in tension. A reasonable modelling of concrete using suitable element type, adequate mesh size, appropriate boundary conditions, realistic loading environment and proper time stepping can help to estimate the governing parameters of concrete. Using these governing parameters (i.e. Poison's ratio, tensile strength, and the stress-strain relationship), the dog-bone tensile specimens are modeled, analyzed and compared the results gathered from experimental outcomes. After evaluation of this parameter by extensive analysis, Finite Element (FE) models showed a good correlation with the experimental results and also showed similar failure patterns. This investigation is intended to validate the FE models with the experimental results by identifying and using the pertinent parameters of the concrete model as well as to provide a successful FE SFRC model for analyzing future problems on SFRC and in context of Bangladesh, the current research aims to investigate the capacity enhancement and stress field of the SFRC from experimental and numerical viewpoint to introduce this new engineering material in the construction industries of Bangladesh.

2. EXPERIMENTAL PROGRAM

In this research, dog-bone specimens are introduced to determine direct tensile strength of plain concrete and SFRC. Two kinds of SFRC and plain concretes, brick (represented as CB in specimen designation) and stone (CS), are tested experimentally and also modelled in FE platform. The fiber volume is taken 1.5% (represented as 1.5 in specimen designation) to cast the tensile specimens. Enlarged end fibers (EE) showed good performance in tensile capacity enhancement compared to straight fibers (ST) and mixed fibers (ES) of small and enlarged ends. The fibers are customized to make enlarged ends for better anchorage. The dog-bone specimens are notched at the middle of the web in four sides which acted as stress concentrator as well as to control the failure location. All the specimens are tested in a 1000kN capacity digital universal testing machine (UTM). Strain data are measured by applying digital image correlation technique (DICT) using high definition (HD) images and high speed video clips and these data are synthesized with the load data from the load cell of UTM which is also followed in the work of Islam (2011), Islam et al. (2011), Uddin et al. (2013) and Dola et al. (2013). The tensile capacity enhancement is found 253%, 204% and 182% compared to control specimen for brick SFRC made of end enlarged fibers, straight fiber and 50-50 mixed fibers respectively. These values in case of stone SFRC are 268%, 175% and 157% respectively. Figure 1 shows the tensile stress-strain behaviour of plain concrete and SFRC made of brick and stone concrete. Dog-bone specimens reinforced with enlarged end fibers are then modelled in the FE platform of ANSYS 10.0 and also validated with the experimental results and failure patterns.



(a) (b)

Figure 1: Tensile stress-strain behaviour of plain concrete and SFRC (a) brick concrete (b) stone concrete

3. FINITE ELEMENT MODELING AND ANALYSIS

All the plain concrete and SFRC dog-bone specimens are modelled on the FE platform of ANSYS 10.0 using an eight node solid element SOLID65. The element is defined by eight nodes having three degrees of freedom at each node: translations in the nodal x, y, and z directions. The element is capable of plastic deformation, cracking in three orthogonal directions, and crushing. In concrete applications, for example, the solid capability of the element is used to model the concrete while the rebar capability is available for modeling reinforcement behaviour. Other cases for which the element is also applicable would be reinforced composites (ANSYS 2005), such as, fiberglass as well as fiber reinforced concrete (FRC). The geometry and node locations for SOLID65 element are shown in Figure 2.

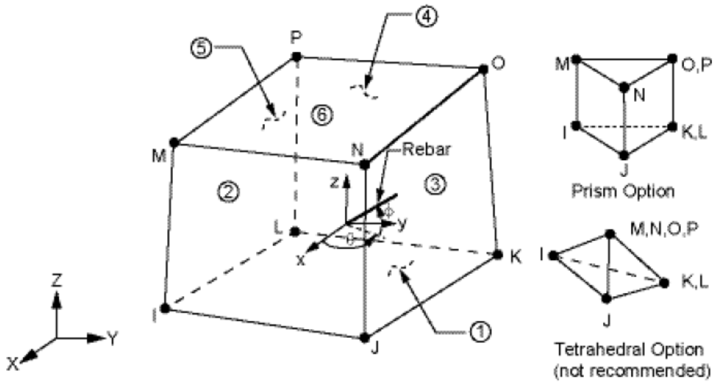


Figure 2: Geometry of SOLID 65 in ANSYS 10.0 platform

The tensile strength of concrete is typically 8-15% of the compressive strength (Shah et. al., 1995). In compression, the stress-strain curve for concrete is linearly elastic up to 30 percent of the maximum compressive strength. Above this point, the stress increases gradually up to the maximum compressive strength. After it reaches the maximum compressive strength, the curve descends into a softening region, and eventually crushing failure occurs at an ultimate stain. In tension, the stress-strain curve (Figure 3) for concrete is approximately linearly elastic up to the maximum tensile strength. After this point, the concrete cracks and strength decreases gradually to zero (Bangash, 1989).

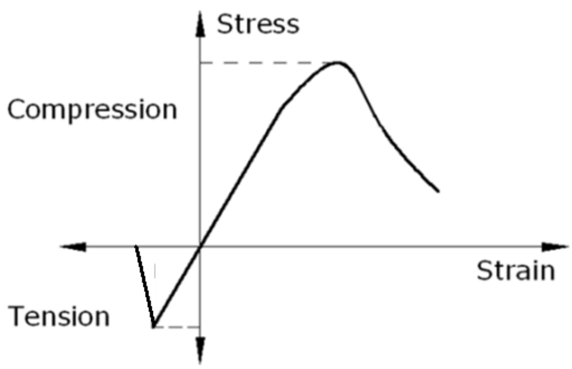
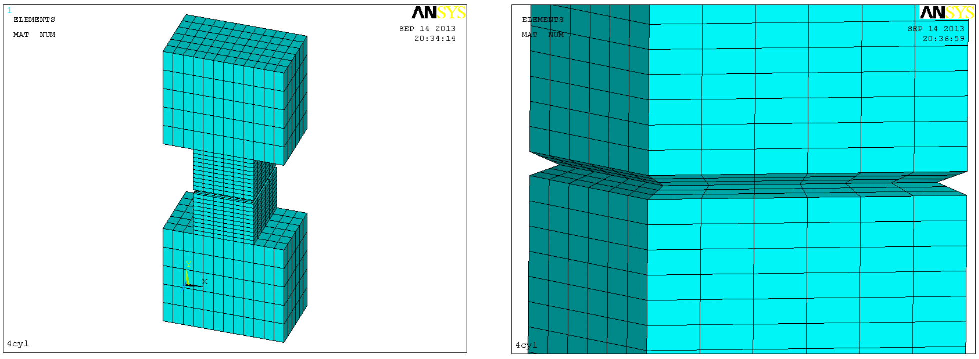


Figure 3: Typical uniaxial compressive and tensile stress-strain curve for concrete.

To model concrete and SFRC in ANSYS platform following properties are to be provided, i.e. (i) elastic modulus, (ii) compressive stress-strain relationship, (iii) ultimate uniaxial tensile strength and (iv) Poisson's ratio. All the values are provided from experimental outputs. Poisson's ratio for concrete and SFRC is estimated to be 0.25 and 0.35 for stone and brick concretes respectively by extensive numerical trials and matching experimental data. William and Warnke (1975) failure criterion is applied to model the concrete as well as SFRC. Four important parameters, i.e. i) shear transfer coefficients for an open crack, ii) shear transfer coefficients for a closed crack, iii) uniaxial tensile cracking stress and iv) uniaxial crushing stress are also considered to model the concretes and SFRC. Typical shear transfer coefficients range from 0.0 to 1.0, with 0.0 representing a smooth crack (complete loss of shear transfer) and 1.0 representing a rough crack (no loss of shear transfer). The shear transfer coefficients for open and closed cracks are determined from the work of Kachlakev, et al. (2001) as a basis.

The dimension of the web of dog-bone direct tension specimens (I shaped with thick flange) are 4x4x5.5 in (100x100x140 mm) attached monolithically with a part of higher stiffness (flange) of size 7x6x4 in (178x150x100 mm) for the gripping the specimens to apply uniaxial tensile force. Loading is applied as displacement boundary condition as it was tested experimentally. The displacement boundary condition is applied in 500 steps followed by 2 sub-steps for each step. Figure 4 shows typical diagram of FE models with boundary conditions.



(a) (b)

Figure 4: FE model of dog-bone specimens (a) full model, (b) close look of notch.

Adequate mesh size plays an important role in FE analysis. To get the accurate result number of element vs. stress graph is plotted and found interesting relationship between them. Element stress changes with the increase in number of element of a specimen, but at a certain ratio stress increasing curve become horizontal. It means after a certain number of element stress will not vary significantly. Figure 5 shows that at a low number of element stress is changing abruptly whereas at a higher number of element reaction became stable, but so much higher number of element will increase the complexicity and also the analysis time of the specimen. A reasonable number of element is selected with maximum accuracy and minimum analysis time. The number of elements taken is 4500 for the FE analyses for dog-bone specimens of both the plain concrete and SFRC.

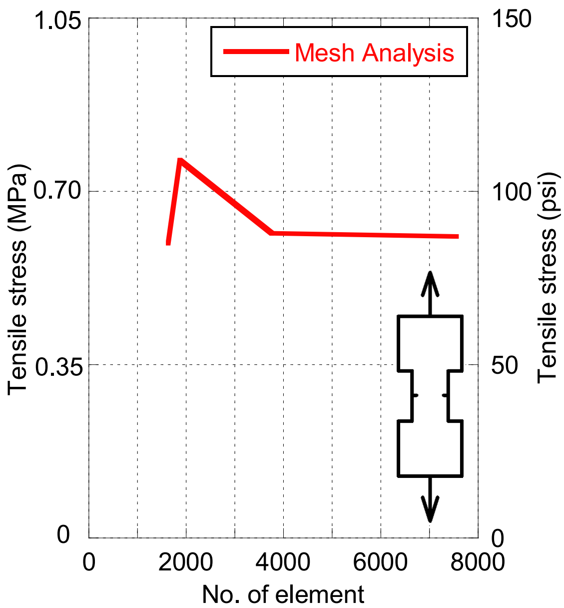
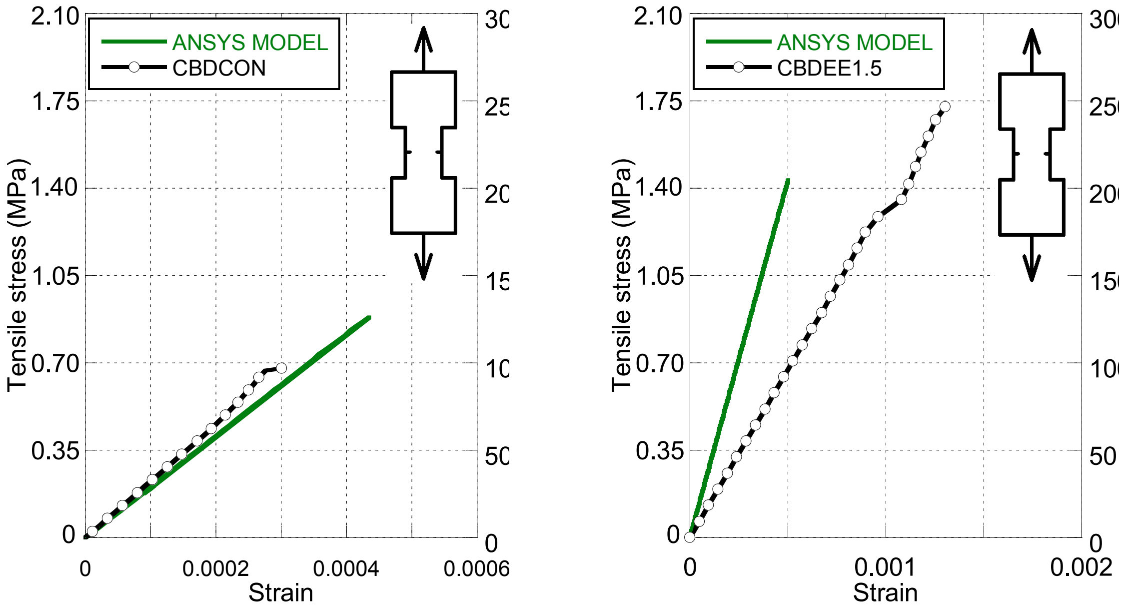


Figure 5: Stress vs. number of elements curve for dog-bone direct tensile specimen.

4. VALIDATION OF FINITE ELEMENT MODELS

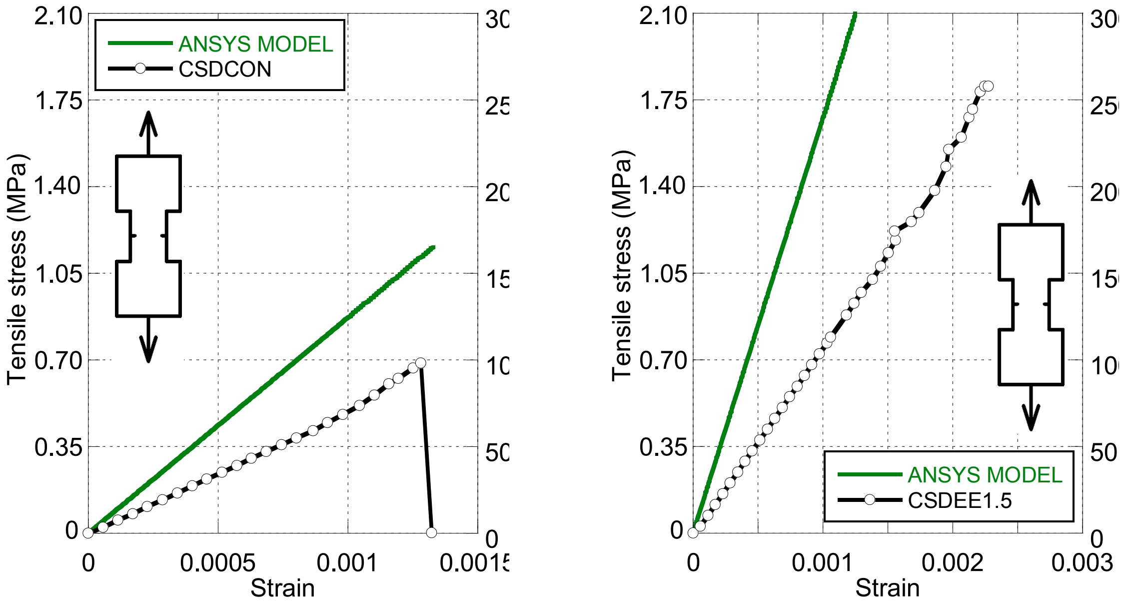
4.1 Validation of tensile capacity

Figures 6 and 7 show the validations of tensile stress results gathered from the experimental measurements with the FE analysis by ANSYS 10.0. It satisfactorily demonstrates the accuracy of the FE model of plain concrete as well as SFRC made of brick and stone concrete respectively. The FE results in most of the cases found to be more or less conservative with respect to the experimental outcomes which also ensure higher factor of safety as well as reliability of the models. In all the cases the increase of tensile capacity is found which is also seen in the experimental results. The accuracy of the prediction and adjustment of the governing parameters helped to validate the models.



(a) (b)

Figure 6: Validation of FE dog-bone specimens with experimental results (a) brick plain concrete and (b) brick SFRC with enlarged end fibers.



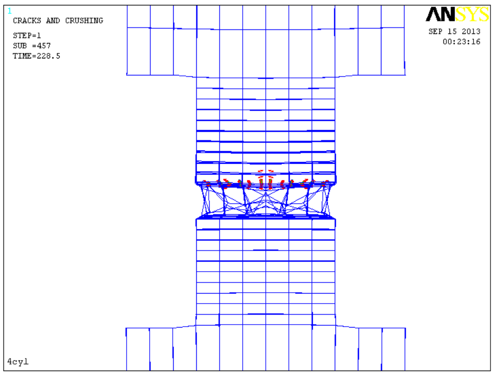
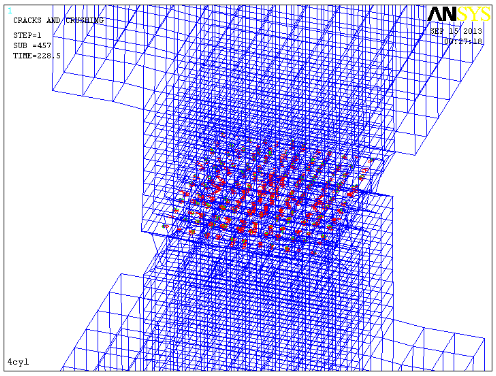
(a) (b)

Figure 7: Validation of FE dog-bone specimens with experimental results (a) stone plain concrete and (b) stone SFRC with enlarged end fibers.

4.2 Validation of failure pattern

Most of the cracks are formed due to tension and at the notch location. Some SFRC specimens showed crack at the joint of web and flange of the dog-bone specimen (clear from the stress contours of Table 1). The crack is found to be a plane fracture surface in experimental investigation which is also found in ANSYS outputs. Plain concrete specimens completely separated in two pieces at the notch but SFRC specimens didn't. Table 1 shows the similarities of failure pattern between the experimental specimen and the FE model using ANSYS. This indicates that the FE modeling of SFRC beam specimens using the pertinent parameters gathered from experimental testing are validated and there remains a good agreement as well as it can be used in future SFRC modeling for tensile loading.

Table 1: Evaluation of failure pattern

<table>
<tr><td>Failed Dog-bone specimen</td><td colspan="2">
</td></tr>
<tr><td>Failed FE model in ANSYS</td><td>
</td><td>
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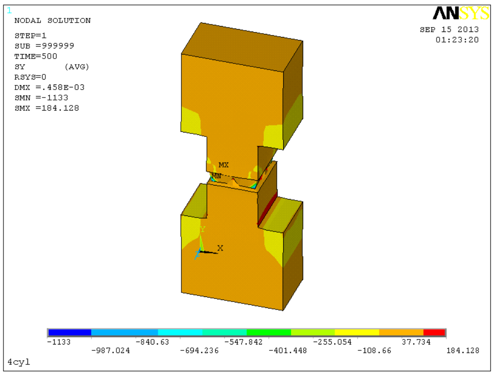
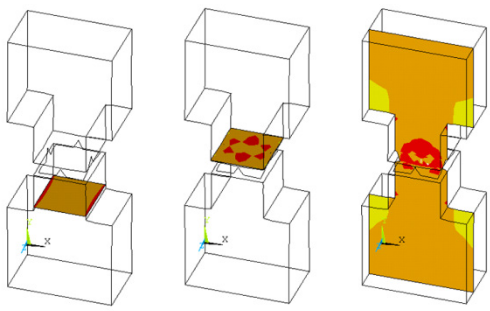
<tr><td>Stress contour</td><td>
</td><td></td></tr>
</table>

5. CONCLUSION

The following conclusions can be drawn from this investigation:

- The tensile capacity enhancement is found 253%, 204% and 182% compared to control specimen for brick SFRC made of end enlarged fibers, straight fiber and 50-50 mixed fibers respectively. These values in case of stone SFRC are 268%, 175% and 157% respectively.

- The performance of enlarged end fibers showed better capacity enhancements.

- The FE models shoed good agreements with the experimental outcomes which represents the validation of the models.

- The failure patterns found from FE analysis is similar to the experimental results which also indicate reliable modeling of plain concrete and SFRC.

- These FE modeling will definitely provide invaluable information of this engineering material to the construction industry of Bangladesh.

REFERENCES

ACI 544.4R-88 (1999). “Design Considerations for Steel Fiber Reinforced Concrete”, American Concrete Institute, Reported by ACI Committee 544.

ANSYS Structural Analysis Guide (2005), Version 10.0, ANSYS, Inc., Canonsburg, Pennsylvania.

Bangash, M.Y.H. (1989). “Concrete and concrete structures: Numerical Modeling and Applications”, Elsevier Science Publishers Ltd., London, England.

Chao, S., Naaman, A. E. and Montesinos, G. J.P. (2009). “Bond Behavior of Reinforcing Bars in Tensile Strain-Hardening Fiber-Reinforced Cement Composites”, ACI Structural Journal/November-December.

Dola, J. F., Islam, M. R., Khatun, M. S. and Hussan, M. (2013). “Investigation of the shear capacity of beams made of steel fiber reinforced concrete (SFRC)”, B.Sc. Engineering Thesis, Department of Civil Engineering, Ahsanullah University of Science and Technology, Dhaka, April.

Islam, M.M., Choudhury, M.S.I., Abdulla, M. & Amin, A.F.M.S. (2011). “Confinement Effect of Fiber Reinforced Polymer Wraps in Circular and Square Concrete Columns”, 4th Annual Paper Meet and 1st Civil Engineering Congress, Civil Engineering Division, Institution of Engineers, Bangladesh (IEB), 22-24 December, Dhaka (Awarded best paper).

Islam, M.M. (2011), “Interaction diagrams of square concrete columns confined with fiber reinforced polymer wraps”, M.Sc. Engineering Thesis, Department of Civil Engineering, Bangladesh University of Engineering and Technology (BUET), Dhaka, August.

Kachlakev, D.I., Miller, T.Y., Chansawat, S. and Potisuk, T.K. (2001). “Finite Element Modeling of Reinforced Concrete Structures Strengthened With FRP Laminates”, California Polytechnic State University, San Luis Obispo, CA and Oregon State University, Corvallis, OR for Oregon Department of Transportation, May.

Lee, S., Cho, J. and Vecchio, F. J. (2011). “Diverse Embedment Model for Steel Fiber-Reinforced Concrete in Tension: Model Development”, ACI Materials Journal/September-October.

Shah, S.P., Swartz, S.E. and Ouyang, C. (1995). “Fracture Mechanics of Concrete”, John Willy & Sons, Inc., New York.

Uddin, M.K., Mustafiz, A. Z., Chowdhury, M. A. and Mondal, P. (2013). “Investigation of the Flexural Capacity of Beams made of Steel Fiber Reinforced Concrete (SFRC)”, B.Sc. Engineering Thesis, Department of Civil Engineering, Ahsanullah University of Science and Technology, Dhaka, April.

Willam, K.J. and Warnke, E.D. (1975). “Constitutive Model for the Triaxial Behavior of Concrete”, Proceedings, International Association for Bridge and Structural Engineering, ISMES, Bergamo, Italy, Vol. 19, pp 1-30, 1975.

CONTENTS (Transportation Engineering)

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DETERMINING SUITABLE BUS STOP LOCATION USING GEOGRAPHIC INFORMATION SYSTEM IN KAPTAI ROAD, CHITTAGONG

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ABSTRACT

In the field of transportation, use of Geographic Information Systems (GIS) is increasing day by day. GIS offers significant spatial analysis by managing data spatially. Since the irrational distribution of existing bus stops in Kaptai road leads to a low public transport service quality thus the study aims to identify the locations of suitable bus stops in order to ensure a better transport system in our study area using a GIS based platform. In this study, determination of suitable bus stop location is based on four criteria: location fulfilling the surrounding maximum public transport demand, land use pattern analysis along study area, available setback from the road ideal for bus Stop shelter and slope. World Geodetic System 84 (wgs84) was used as the datum. Origin-Destination survey was conducted to get the locations of the existing bus stop and to identify the probable trip generation and distribution zone. Thus a digitized geo referenced transit routes layer as line features, slope map derived from Digital Elevation Model (DEM) and 30 feet buffered from road center layer when overlaid on a land use layer we can analyze the socio economic characteristics of the area surrounding transit route and bus stops. Finally using the stated criteria for classifying the bus stops location, there are six locations has been considered as very good bus stops denoting sites are free from any use, meet the optimum surrounding public transport demand and situated relatively flat land. The evaluation of identified bus stop location indicates that redundant stops in study area will be reduced and service coverage will be enlarged by distributing fewer bus stops. The conclusion was that by giving bus stops location a thorough consideration would help in enhancing public transport system by boosting the principles of good access which is Safety, Affordability, Accessibility and Reliability.

Keywords: *Public transport demand, GIS, Bus stop location, overlay, Kaptai*

1. INTRODUCTION

1.1 Background of the study

Chittagong City serves as the port and commercial capital of Bangladesh. The City and it hinterland has a regional success story in past year. In recent years the growth of population in the City has been increasing faster than other cities of Bangladesh. The City currently has a population of 2.6 million (BBS, 2011) whereas Chittagong Development authority (CDA) projected that the city population will increase 3.38 million in next 2015 year (DAP, 2008).This rapid growth rate is the cause of increasing labor force participation rates. Total employments have also exceeded into the cities at different location which resulting extra pressure on transport system. To reduce this pressure, inner roads and the outer road that connected with the city need to be well-functioning and efficient to confirm better communication. But transportation system of Chittagong has not kept pace with large amount of road users and numerous motor vehicles. Governments and respective institutions are also facing the problem of maintaining through traffic and safety on highways. So the City is experiencing transportation problems and levels of congestion day by day. (Debnath and Islam, 2009). Lack of bus stoppage is a major problem which is responsible for unwanted road congestion and suitable location of the bus stop along the road side is very important to obtain a successful traffic management.

1.2 Statement of the problem

Chittagong to kapati is one of the major regional roads of the Chittagong district which carry more importance both in regional and national basic. The section from Raster matha to CUET gate of this road also carry the same importance by connecting with number of tourist place, Chittagong University of Engineering and Technology, timber mills, defense headquarter and other national organization and office. Total length of the road is nearly 15 km. People of this region make a numbers of trips to the city travelling by this major road for job or various purposes at different period of time of a day. There is lack of bus stoppage along this road and the buses do not maintain the rules to stop at a certain location for trip generation and distribution. So people from far region wait in a scattered way to the area where buses stop generally. This is the cause of unwanted congestion of this road that hamper free flow of traffic. This report will find out suitable bus stops location along the Raster matha to CUET gate section of Chittagong to Kaptai major road and will convenient to the people of this region in making trip to the city and other location from a certain place. It will also help full for ensuring free flow of vehicles and other traffic avoiding unwanted accident.

1.3 Aim and objectives

Determining Suitable bus stop location using GIS along Raster matha to CUET gate section of Chittagong to Kaptai major road is the aim of this study. For fulfilling this aim, some potential bus stop locations were determined with its approximate catchment area based on maximum trip generation and distribution rate. This locations were Justified considering land use pattern analysis, slope analysis and available setback from the road ideal for shelter.

2. STUDY AREA

Kaptai road a regional road lies under the administrative jurisdiction of Chittagong district. It is located far from the center of the city. This road started from the place called “Raster matha” and end to the place called “Navy Camp”. But in this research it covers area up to 18 kilometers from Raster matha to CUET gate. The main consideration would be to determine the suitable locations for bus stop along this road. The road section consists of various road intersections, bus stoppage, major trip generators, restaurant, industries, growth centers, religious places, schools, defense quarter, and government institutions etc. which bring out the importance of sufficient bus stoppage along this road. Geographically, this road subjected to annual precipitation. It is non-flooded region. It is hardly happened when the amount of precipitation is high in comparison with other years, rain water sometimes logged on the road for a short duration.

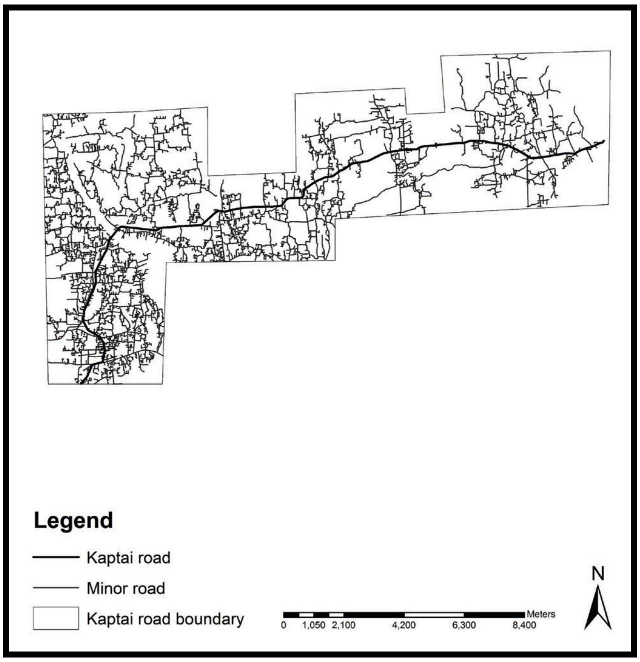

Figure 1: Study area

3.

4. METHODOLOGY

This research involves origin-destination survey to Kaptai road area for specific sample to establish trip generation and distribution data. Some relevant information about study area was also obtained through secondary source such as GIS shape file for existing road and structure from Chittagong Development authority. After accumulating primary and secondary data, survey data are being analyzed through Microsoft Office Excel. This step is mainly focused on determining primarily potential bus stop location based on trip generation and distribution rate and finally showed the catchment area of those potential site. Data Pre-Processing is the first step of GIS based analysis involves digitizing geographical spatial features and extracting features. After that preparing suitability map is the next step involves justifying primarily potential sites through land use pattern analysis, slope analysis and determining available setback along road side.

Land use pattern analysis: Main concern of this step involves land use condition analysis of primarily suitable location whether the identified area has the influence of economy, society and also demarcating the existing demand.

Slope analysis: Slope is derived from the Digital Elevation Model (DEM) which was derived from elevation values. The elevation values were extracted at 30m interval so as to reduce interpolation. DEM and slope were generated in raster format, the slope was reclassified into three (3) classes i.e. Good, moderate and bad. Main objective of this step is to determine the less sloped surface ensuring safety for bus stop location.

Available setback: The available setback from the road ideal for bus stop shelter is a polygon feature which was achieved by overlaying the digitized structure by the road side with the setback created 30 feet away from the center of the road. Main concern of this step involves making room for space where any further development beside the road can be located to enhance the flow of traffic.

Overlay analysis: By overlying of these three layers give suitability map for Bus Stop location along Kaptai road

4.1 Sample size determination for Origin-Destination survey

A complete list of employed person of surrounding Kaptai road area has been used for the determination of sample size. The initial sample size was determined by the following formula:

$$n = \frac{Z^2pq}{E^2}$$

Where,
Z is the normal variate and which has 1.96 for 95% confidence interval
P is the target proportion.in this case we assumed p=0.5
P+q=1, therefore q=.50 and E is the desired error which is .7
The initial sample size is therefore:

$$n_0 = \frac{(1.96)^2 * .5 * .5}{(.07)^2}$$
$$=196$$

These sample size was adjusted by using the following formula:

$$n = \frac{n_0}{1 + \frac{(n^0 - 1)}{N}}$$
$$=193.64$$

Where n is requiring sample size and N is complete list of employed person. The sample size has been distributed among the study area. Here 200 samples are taken for the formulation of this study.

5. RESULTS

5.1 Findings from origin-destination survey

Origin-Destination survey for study area has been conducted during both peak and off peak hour. Survey data were collected from different transport mode users like CNG, bus, truck and car users. They were asked to inform about their origin-destination point and distance of their settlement from origin-destination point. From this survey primarily seven sites were selected as potential bus stop location for Kaptai road. The results are given below:

Table 1: The overall trip generation and distribution rate of Kaptai road

Potential Bus stop location	Trip generation (%)	Number of interviewed passengers(Trip generation)	Trip distribution (%)	Number of interviewed passengers(Trip distribution)
Raster Matha	9	18	8	16
Kuaish	7	14	5	10
Nozumia hat	11	22	4	8
Niamotali	3	6	3	6
Modunaghat	10	20	9	18
Nowapara	23	46	22	44
Pahartali	8	16	9	18
CUET gate	9	18	10	20
Bahaddarhat	20	40	30	60
Total	100	200	100	200

Source: Field survey, 2013

The above table shows that Nowapara intersection is the most potential site for bust stop location based on trip generation and distribution rate. Then Nozumia hat, Modunaghat, raster matha, CUET gate, pahartali, and kuaishare are the rest of the potential sites respectively. Catchment area belongs to each of the location varies from 2-5 km. Bahaddarhat intersection is not to be considered for potential location for bus stop as it is excluded from study area.

5.2 Justification analysis of primarily suitable location

5.2.1 Land use pattern analysis

Land use in study area are mainly categorized into eight types including residential, commercial, community service, service activity, education and research, manufacturing and processing, agriculture and mixed use. Here approximately 14751 number of structure has been found (DAP, 2008).

Land use along Rastarmatha, Kuaish and Nozumia hat Moore

The road through rastar matha intersection is mainly connected with Kalurghat area whereas Kuaish Moore is connected with Oxygen Moore, Kaptai road and nozumia hat moore are serving as growth center also connected with kaptai road. As a result here residential and commercial places are mostly dominated land use type. Traffic congestion is high and large number of shops, commercial institutions, bank which are available in this area. Quaish College and some other educational institutions are situated in this area which leads to high residential density. Quaish Moore is the only place from where people along kaptai road can easily travel to main city part. Large number of people from Mohora, Burishchar, CMB, waizdia, and Maizpara come here due to commercial purpose. As nozumia hat moore is serving as growth center so it is a vital transport point belongs to the people of this area. Number of garments and light industries are situated around rastarmatha moore, so every day thousands of people use this intersection point spontaneously.

Land use along Modunaghat Moore

Modunaghat bridge on Halda river connecting Chittagong city part and Kaptai is situated at this point in where large area of community has been developed. Residential density is very high and several types of commercial activities are done in this area. Kacha bazar, shopping market, bank and some other educational institution lead to high congested area. Recently one treatment plant of CWASA is building up just beside the intersection point and also a power plant station is located here. . People along kaptai road can easily travel to Hathazari area using this point. Due to high service and commercial potentiality every day lots of people come here from Niamotali, hazipara, Tendolerghat, Akbaria, Barua, urkir char, zia bazar.

Land use along Nowapara Moore

Nowapara is one of the vital intersection points along kaptai road and serving as growth center. The area has a great potentiality for lots of commercial activity where numbers of shopping center, kacha bazar, Bank, light industry are quite available. Nowapara Degree College and some other educational institutions are located just around this intersection point leads to high residential density. People along kaptai road can easily travel to

Rawzan by using this point. Many supporting industry like ice factory, small bakery etc. are greatly serving to some other medium industry around Nowapara intersection point. Every day thousands of people came here from Chittagong city, Rawzan, Middarpara, Sham mahaldarpara, Komolardighi, Adharmanik, Mongolkhali for various purposes.

Land use along Pahartali Moore and CUET gate

Pahartali Moore is serving as a vital growth center point along kaptai road where number of commercial institution, Bank, Kacha Bazar, medical center, restaurants are quite available. Here one renowned engineering university named Chittagong University of Engineering and Technology (CUET) is situated resulting many students are using this point everyday for going Chittagong city in fact it is the only point to travel city part. Besides CUET school and college and some other educational institutions are also located. During hat day of Gaurishankar, Pahartali intersection point is greatly used. Recently some residential housing projects, shopping complexes are building up around this moore due to high commercial demand. People can easily travel to Rawzan and Betagi by using this intersection point. Every day lots of people from Betagi, Rawzan, Rangunia, Kaptai, Goshchi, Unoshhottor para come here due to various purposes.

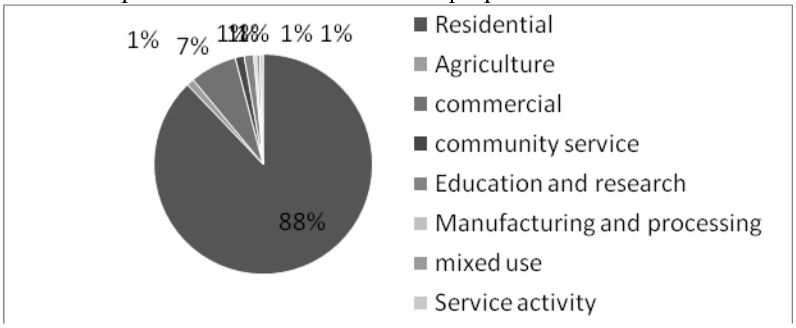

Figure 2: Area coverage by different types of Land Uses along Kaptai road
Source: DAP, 2008

From figure 2 it is clear that about 88% of land along Kaptai road is used as residential purpose and 7% area is used for commercial activity.

5.2.2 Slope analysis along Kaptai road

The elevation values of study area are generated at 30 m interval. The lowest elevation point value is 0m leveled with sea level while the highest elevation point value is 50 m above sea level. The location of each elevation point is represented by its X and Y i.e. Easting and Northing Coordinate respectively. From the DEM, Slope was derived; the output measurement was set to degree. As stepper land considered as bad and flat land considered as good for bus stop location, the slope map was reclassified into three (3) classes i.e. bad (20-55), moderate (10-20) and good (0-10) as shown in figure for a proper analysis. Bus Stops should not be located on sloped surfaces to ensure safety. This gives us a slope suitability map of bus stops locations along kaptai road. In elevation map it was found that Rastar matha intersection point has the highest elevation value varies from 40-50m, kuaish and Nozumia hat intersection point belongs to 1-3m and rest of the intersection point varies from 3-5m high from mean sea level. In slope suitability map primarily potential location has been identified as they are located at less sloped surfaces meaning good sloping condition.

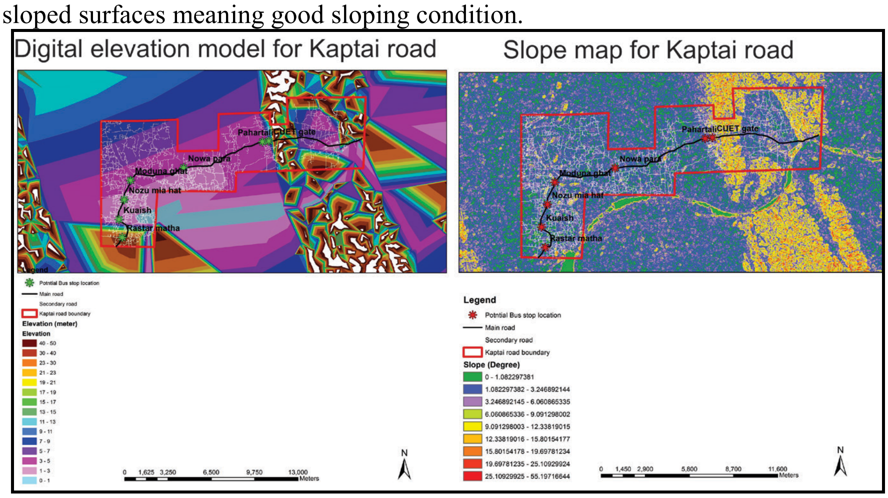

Figure 3: DEM and slope map for Kaptai road
Source: Analysis by authors

5.2.3 Available Setback from the road ideal for shelter

In planning, it is stipulated that a reasonable space should be left between the road shoulder and any form of development. The space between the road shoulder and any development is known as buffer zone or setback. It is important so that when the road is to be expanded or when government wants to provide certain services such as bus stops, the government would not need to pay compensation for legal owners of the land. Kaptai road like many other places in urban Chittagong is faced with that problem where there is little or no setback from the road shoulder. This has restricted where bus stops can be located because a road should have setback so as to avoid congestion which will enhance the flow of traffic. As the road is maximum 30 feet wide and thus for providing minimum 12 feet width bus stop with suitable walking space in this research, structures along the road side were digitized as polygon feature and the road map was buffered at 30 feet from the center of the road to make room for space where any further development beside the road can be located to enhance the flow of traffic. An overlay analysis of the polygon features was carried out using the merge option and classification was used to distinguish different features. The polygon features are the buildings along the road side and the 30 feet buffer of road network. Places where the digitized residential polygon falls inside the 30 feet road buffer denotes that there is no space in that particular location where bus stop can be located without the bus stop serving as hindrance to the flow of traffic. From figure 4 it will be clear that location around Rastarmatha moore, Quaish more, Nozumia hat moore, Modunaghat moore, Nowapara moore, Pahartali moore and CUET gate provide suitable setback provision where further development like Bus stoppage, seating arrangement can be easily established. Table 2 shows that all primarily potential location has available space for bus stop.

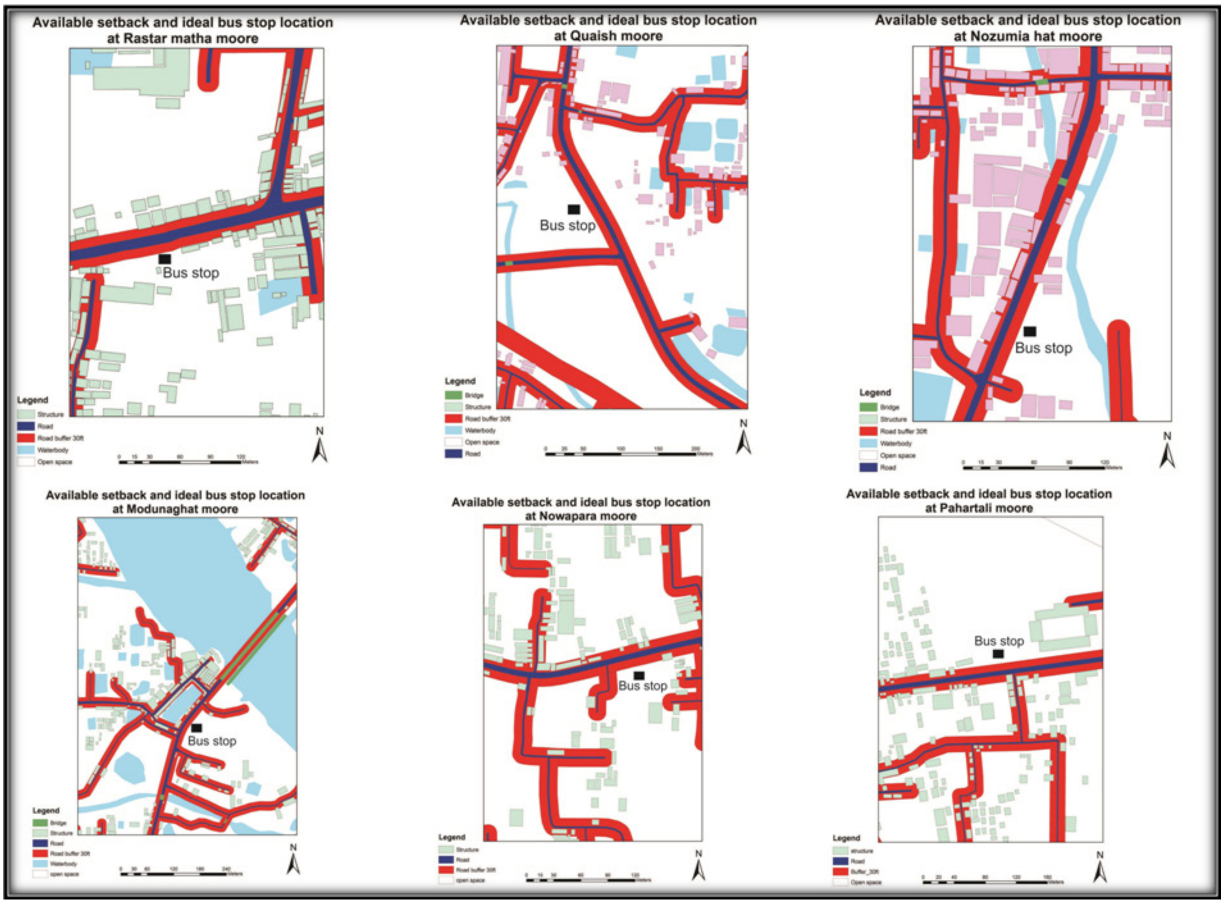

Figure 4: Available setback and ideal bus stop location for potential site
Source: Analysis by authors

Table 2: Distance of suitable bus stop from intersection point

Intersection point	Distance of suitable bus stop from intersection point (feet)
Rastar matha	50 (west)
Quaish	10 (North)
Nozumia hat	75 (South)
Modunaghat	20 (South)
Nowapara	20 (East)
Between Pahartali moore and CUET gate	70 (East from Pahartali and west from CUET)

Source: Field survey, 2013

6. RECOMMENDATION:

6.1 Summary of findings:

Overlying the slope map, setback map and also considering land use pattern and trip distribution- generation data some suitable location has been recommended. They are Rastar matha, Quaish more, Nozumia hat, Modunaghat, Nowapara and between Pahartali and CUET gate. Table 3 will show the sum up result of total research:

Table 3: Summary of findings

Intersection point	Distance of suitable bus stop from intersection point (feet)	Available free space(length in feet)	Slopping condition	Elevation (from mean sea level)	Potentiality (Economic, social and traffic demand)
Rastar matha	50 (west)	150	Good	40-50m	High
Quaish	10 (North)	250	Good	1-3m	High
Nozumia hat	75 (South)	150	Good	1-3m	High
Modunaghat	20 (South)	200	Good	3-5m	High
Nowapara	20 (East)	180	Good	3-5m	High
Between Pahartali moore and CUET gate	70 (East from Pahartali and west from CUET)	200	Good	3-5m	High

Source: Field survey, 2013

6.2 Bus bay dimension:

Considering Bangladeshi perspective, some modified recommendation are given below based on "Guidelines for the Location and Design of Bus Stops for Washington, DC"

- Stopping area length consist of 35 feet for each standard 25 feet bus
- Bus bay width desirably 12 feet. For traffic speed under 30 mph, a 10 feet minimum bay width is acceptable. These dimensions do not include gutter width.

- Suggested tapper width are listed given to the table below.
- Minimum design for a bus bay does not include acceleration and deceleration lanes. Recommended acceleration and deceleration length are listed below.

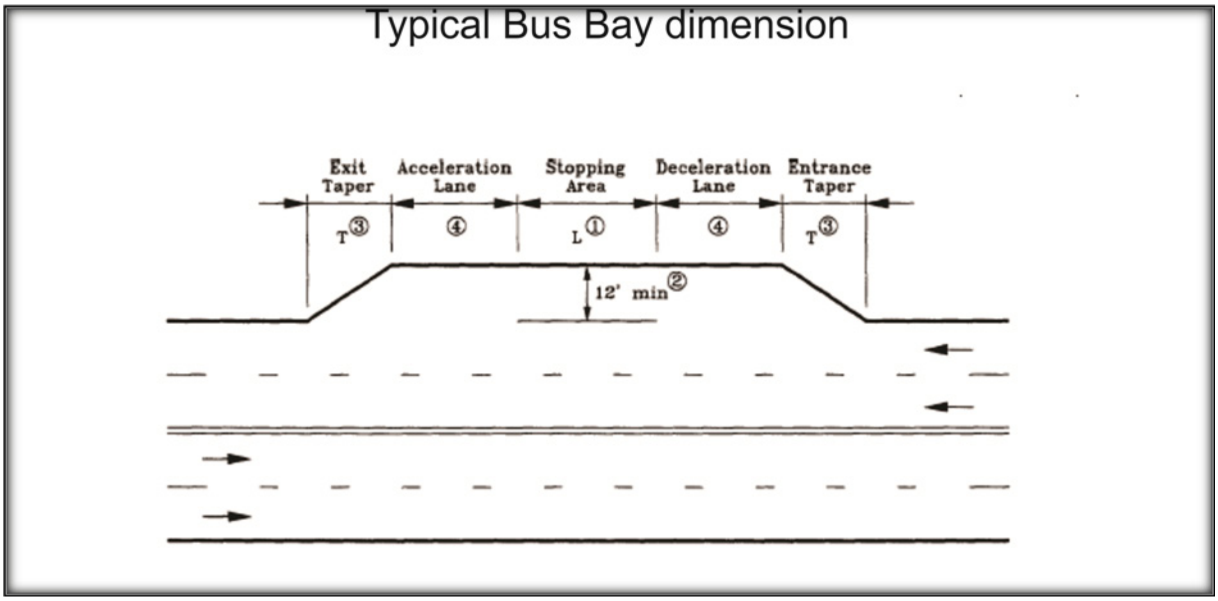

Fig 5: Typical Bus bay dimension
Source: Fitzpatrick, 1996

Another alternative to the bus bay design is a partial open bus bay (or a partial sidewalk extension) has been recommended in this study. This alternative allows buses to use the intersection approach in entering the bay and provides a partial sidewalk extension to reduce pedestrian street-crossing distance. It also prevents right-turning vehicles from using the bus bay for acceleration movements.

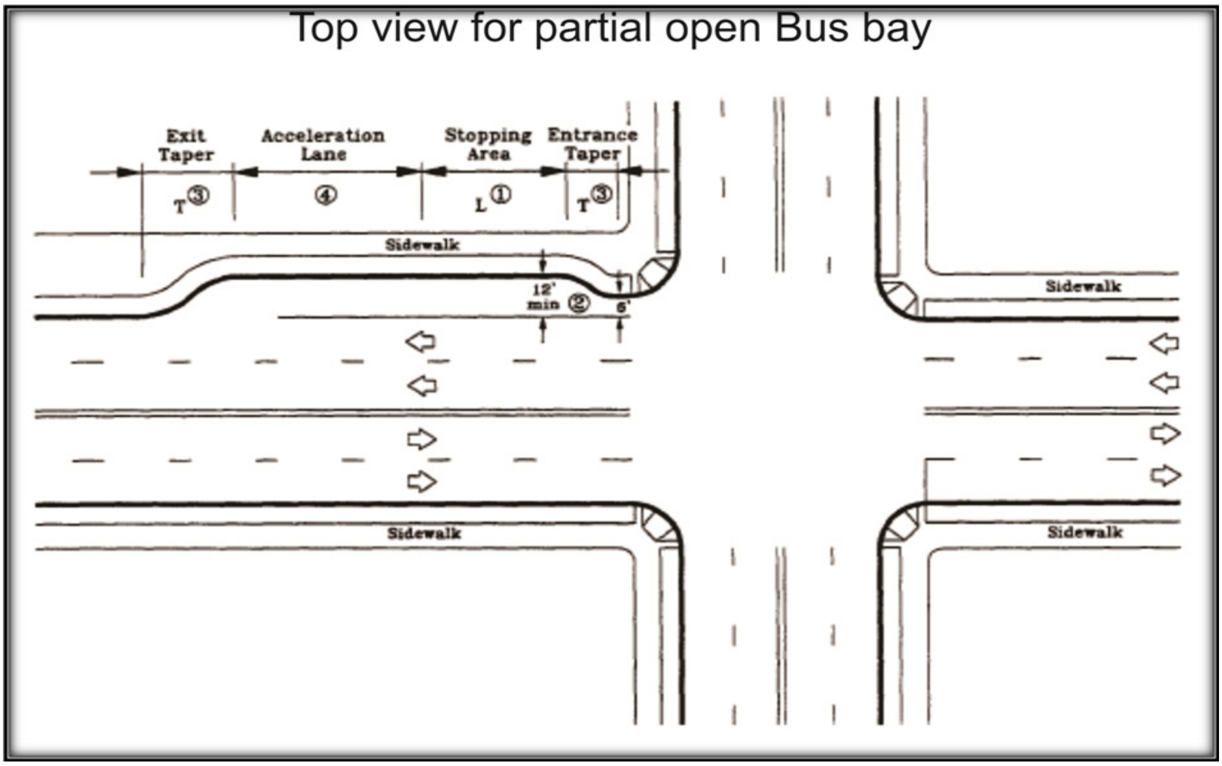

Fig 6: Partial open bus bay
Source: Fitzpatrick, 1996

7. CONCLUSION:

This research work is an approach of GIS-based methodology to identify bus stops in selected study area which are identified based on three attributes; these are location, characteristics, and surface (located on a sloped surface). The result of standard uniform spacing was shown and if this is implemented it will give a level of attractiveness and orderliness to bus services in the study area. Geographical Information Systems cuts across many professions. The study has demonstrated the spatial analysis capability of GIS technology in identifying best location and pointers to an articulated decision making in public transport route management. Therefore applying this procedure to locate bus stops will help in modeling spatial problems for decision makers and will also proffer solutions to different forms of spatial issues affecting man and his environment. Finally, adopting this methodology, would accomplish the principles of good access which are Safety, Affordability, Accessibility and Reliability in public transport in the study area.

8. REFERENCES

Debnath, R and Islam, M.S., (2009). "Managing efficiency of an urban road toward better transport system", http://academicjournals.org/journal/JGRP/article-full-text-pdf/16399851077

Metro, (2009). "Guidelines for Bus Stop Design, Information and Placement, Customer Service, Operations and Safety Committee", Washington Metropolitan Area Transit Authority, http://www.wmata.com/about_metro/board_of_directors/board_docs/031120_3ABusStops.pdf

Fitzpatrick, K. H., (1996). "TCRP Report 19: Guidelines for the Location and Design of Bus Stops", http://gulliver.trb.org/publications/tcrp/tcrp_rpt_19-d.pdf.

Bangladesh Bureau of Statistics. Population Census 2011, Community Series, Zilla: Chittagong

Chittagong Development Authority, (2008). Detailed Area Plan (DAP) for Chittagong. (Chittagong: CDA-2008)

ESTIMATION OF THE HIGHWAY CAPACITY IN KHULNA METROPOLITAN CITY

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ABSTRACT

The adequacy or sufficiency of existing highway networks in service to current traffic will not be known until an accurate value of basic capacity is used in capacity analysis procedures. This study was undertaken as a stepping stone towards the determination of the basic capacity of interrupted flow highway facilities in Khulna – Jessore National Highway, Bangladesh. The study involves investigating the characteristics of the actual time headways (spacing) adopted by drivers in the traffic stream under prevailing conditions. The reciprocal of the minimum time headways adopted by passenger cars under capacity flow conditions has been used in the estimation of the basic capacity. Comparisons are made between capacity values based on 5-minutes and 15-minutes mean time headway. In addition, comparisons of 5-minutes and 15-minutes equivalent hourly flow rates are made. The effects of the size of the vehicles and the lane use on the capacity have been investigated. Passenger car equivalents (PCEs) for medium and large sized vehicles have been calculated. Results of the study indicated that the value of basic capacity was 575 passenger cars per hour per lane at an average speed of 40 km/h. Comparison with the equivalent hourly flow rate approach yielded a capacity value of 498 passenger cars per hour per lane. PCEs for medium and large sized vehicles were obtained as 1.00 and 2.43, respectively.

Keywords: Capacity, Headway, Passenger car equivalents, Interrupted flow.

1. INTRODUCTION

The rapid increases in traffic volumes and congestion on the streets and highways in Khulna metropolitan city have posed a serious problem whose solution requires the construction of costly roadways. The primary reason for constructing highways to serve traffic and a study of highway capacity is a study of the effectiveness of the various facilities in serving traffic. The selection of highway type and its dimensional needs such as the number of lanes and the minimum length of weaving sections cannot be obtained without first obtaining information as to the capacity of the highway. Isolation of bottlenecks and preparations of estimates of operational improvements that may be expected to accrue from prospective traffic control measures or from spot alterations in highway geometry can only be undertaken after utilizing highway capacity information (AASHTO, 1984).
Capacity of a highway is the maximum hourly rate of flow of vehicles that can be accommodated by it under prevailing conditions. Prediction and knowledge of capacity are fundamental in design, planning, operation and layout of road network sections. Until an accurate value of basic capacity which is applicable to the highways of a particular nation is obtained, capacity analysis of highways cannot be conducted properly and the time in the future when traffic growth may result in an undesirable degree of congestion will not be known. Due to this reasons this study is carried out for the development of basic capacity in Khulna – Jessore National Highway.
The specific objectives of this study are:

- To examine the effect of the sizes of vehicle on the capacity of highways.
- To examine the effect of lane use on the capacity of highways.
- To estimate the passenger car equivalents for medium and large sized vehicles.

2. METHODOLOGY

2.1 Study Site

In this study Khulna – Jessore National Highway (NH7) at Daulatpur was selected for this study. This particular highway had the following characteristics:

- It comprised two-lane and single carriageway sections.

- There was heavy traffic throughout the day.
- At the time of this study trucks and buses were permitted to use the lanes.
- Motorcycles and three-wheeled motorized vehicles were permitted to use the lanes.
- The lanes were 3.5 meters wide, a shoulder which was 1.25 meters wide was adjacent to the shoulder lane.

2.2 Data Collection

Due to the constraints of cost and time, only one location on the Daulatpur of Khulna –Jessore National Highway was considered in this study. As this study involved investigating the effect of the sizes of vehicles on capacity, data was collected so as to include different periods of flow as shown in details of the data collection schedule in Table 2.1.

Table 2.1 Dates and Times of Data Collection

Date	Time
Monday , 13.05.2013	4.00 pm – 5.00 pm & 5.05 pm – 6.05pm
Wednesday , 15.05.2013	8.00 am – 10.00 am & 11.30 am – 1.00 pm

The photographs in Figure 2.1 have been included to aid in the description of the geometric characteristics of the facility.

(i) View from top

(ii) View from roadside

Figure 2.1 Traffics at Study Site.

2.3 Method Of Analysis

To perform the analysis of the highway capacity, data had to be extracted from the video recordings of traffic flow on the different days different time periods. All the video memory was then re-recorded on new sets of computer while a video timer was simultaneously employed to dub the time in seconds. The time appeared on the television screen when the video clips were played.

2.3.1 Time Measurement

Data was extracted in the form of time headways defined as the time in seconds for the rear bumper of two successive vehicles in the same lane to pass a single, common reference mark. Data was also gathered so that the times taken by each vehicle to traverse the known distance delimited by the reference marks was subsequently used to compute the time mean speed of each vehicle.

2.3.2 Classification Of Headways

For the purpose of this study, vehicles were classified into three major types: small (S), medium (M), and large (L) vehicles. All passenger cars were classified as small-sized vehicles (S); vehicles which were of a larger size than passenger cars such as vans or pickups which had only four wheels were classified as medium-sized vehicles (M) and all other vehicles which were larger than medium-sized vehicles were classified as large-sized vehicles (L). Nine different headway types were classified and coded as:

1. Small-sized vehicle followed by small-sized vehicle (S-S).

2. Small-sized vehicle followed by medium-sized vehicle (S-M).
3. Small-sized vehicle followed by large-sized vehicle (S-L).
4. Medium-sized vehicle followed by small-sized vehicle (M-S).
5. Medium-sized vehicle followed by medium-sized vehicle (M-M).
6. Medium-sized vehicle followed by large-sized vehicle (M-L).
7. Large-sized vehicle followed by small-sized vehicle (L-S).
8. Large-sized vehicle followed by medium-sized vehicle (L-M).
9. Large-sized vehicle followed by large-sized vehicle (L-L).

3. ANALYSIS AND DISCUSSION OF RESULTS

A comparison is made in this study, for all the observation periods between values obtained by using flow rates and mean time headways for 5-minute and 15-minute periods. Results of the study of equivalent-hourly flow rates are contained in Appendix A. In order to obtain an accurate value of average minimum time headway, a headway analysis was conducted of the data of the entire data set and the results are documented in Appendix B.

3.1 Capacity Based On Equivalent Hourly Flow Rates

The methodologies of the 1985 HCM do not generally deal with full hour volumes, but rather with equivalent hourly rates of flow during a peak 15-minute interval within the analysis hour. The peak hour of flow during the courses of this study was from 5:05 pm to 6:05 pm with a volume of 514 vph. From Table A.4 of Appendix A it can be seen that that peak 5-minute period within the analysis hour occurred from the 5:20 pm to 5:25pm with a volume of 48 vehicles. This yields an equivalent-hourly flow rate of 576 pcphpl on the right lane. It can also be observed that the peak 15-minute period within the analysis hour, obtain by summing up 3 successive 5-minute periods, occurred from 5 : 15 pm to 5 : 30 pm with a volume of 126 vehicles. This provides an equivalent-hourly flow rate of 504 pcphpl.

3.2 Comparison Of 5-Minute And 15-Minute Rates Of Flow

A comparison of the peak 5 and peak 15-minute rates of flow was conducted for the four periods of observation made during the course of this study on a lane-by-lane basis. Previous studies (1985 HCM and Roess and McShane, 1987) have stressed that the 5-min rates of flow where statistically unstable and HCM based its analysis procedures on 15- min rates of flow. The results of the comparison between 5 and 15 – min rates of flow are shown in Table 3.1.

Table 3.2 Comparison of 5 and 15 – Minute Rates of Flow

Time	Lane	Rate of Flow (pcphpl)		
		5 min	15 min	% diff
8.00 am – 10.00 am	Left	408	376	7.84
	Right	432	364	15.74
11.30 am – 1.00 pm	Left	516	480	6.90
	Right	492	448	8.90
4.00 pm – 5.00 pm	Left	420	392	6.67
	Right	360	368	2.18
5.05 pm – 6.05 pm	Left	552	498	9.88
	Right	576	504	13.88

Note: The values of peak flows are shown in Appendix A.

The results show that average difference in the value of the equivalent-hourly rates of flow for the left and right lanes are 7.82% and 10.20%, respectively. The overall value for the combination of all the lanes is 9.00%.
The results indicate that the greatest variability was in the right lane with values ranging from 2.18% to 15.74%. The difference is quite substantial due to the combination off-peak flows and flows approaching capacity.

3.3 Comparison Of Average Headways For 5-And 15-Minute Periods

Details of the results of the headway analyses are documented in Appendix B. the difference between the average minimum time headway for peak 5- and 15-minute flow periods and a comparison of the values of capacity for the mixed traffic stream is summarized in Table 3.2

Table 3.3 Comparison of 5 and 15 – minute Headway and capacity

Lane	Time	Headways (sec)			Capacity (vph)	
		5 min	15 min	% diff	5 min	15 min
Left	8.00 am – 10.00 am	9.20	10.10	8.90	392	357
Right		8.40	9.70	13.40	429	372
Left	11.30 am – 1.00 pm	7.40	7.80	5.12	487	462
Right		6.48	7.25	10.62	554	497
Left	4.00 pm – 5.00 pm	8.80	8.95	1.67	410	403
Right		9.45	9.60	1.60	381	375
Left	5.05 pm – 6.05 pm	6.30	6.90	5.80	572	522
Right		6.40	7.15	10.48	563	504

It can be seen that the difference between the mean time headways for peak 5- and peak 15-minute time period when averaged for all the lanes is 7.2%. The corresponding value of capacity is also seen to difference by 7.2% when the average of all the lanes is considered. The high difference observed is partly due to the inclusion of observations of headways during periods of low flow.

3.4 Capacity Based On Average Minimum Time Headway

The basic capacity of the National Highway No.07 site can be estimated by the reciprocal of the mean time headway between small-sized vehicles.
Table 3.3 summarizes the values of average minimum time headways adopted by small-sized vehicles following small-sized vehicles during the peak hour of flow considered for both 5- and 15-minute periods.

Table 3.4 Summary of Headway Observation

Item	Duration of peak flow	
	5 min	15 min
Mean headway (H)	6.37	6.67
Standard deviation	2.35	1.83
95 percent confidence interval for the mean	4.79 < H < 8.15	5.00 < H < 8.40

From the mean headway estimated for small-sized vehicles followed by small-sized vehicles (S-S) headway type, for the peak 15-minute flow on the left lane with large-sized vehicles present, the 95% confidence interval for the maximum volumes is estimated to be 720 to 429 with a mean of 575 passenger cars per hour (pcph). From Appendix B (Table B.4), it can be seen that for this capacity flow average speed is about 40 km/h.
In order to be on the conservative side while estimating the value of capacity, the value of capacity obtained by considering the headway for the 15-minute flow is considered to be the representative value of this study.

3.5 Effect Of Lane Use On The Capacity

In the study site the both lane was used to large sized vehicles. From the lane distribution by vehicle type, obtained by considering all the vehicles observed throughout the course of this study, as shown in Table 3.4

Table 3.5 Lane distribution by vehicle Type

Vehicle Type	Percent Distribution by Lane	
	Left	Right
Small	71.21	67.40
Medium	17.89	17.18
Large	11.00	15.42

The average headways, as shown in Table 4.2, reveal obvious differences between the values of capacity obtained for the different lanes. The lane with the higher proportion of large-sized vehicles is seen to have the larger headway. These findings are in agreement with Gywnn (1968) who observed that the percentage of trucks in a traffic lane had an effect on the average headway of all vehicle sizes and that the headways increased with an increase in the percent of trucks. Thus, the capacity of a traffic lane is seen to be reduced as predicted by the 1965 HCM due to the presence of large-sized vehicles in the traffic stream. This is rather obvious if comparisons are made with the values of capacity shown in Tables 3.5 and 3.6 as obtained by considering the flow rates and the average time headway for the left and the right lanes respectively.

Table 3.6 Comparison of Capacity by Lane (Based on Flow Rates)

Time	Lane Capacity (pcphpl)		% difference Left Vs. Right
	Left	Right	
8.00 am – 10.00 am	376	364	3.3
11.30 am – 1.00 pm	480	448	6.6
4.00 pm – 5.00 pm	392	368	6.1
5.05 pm – 6.05 pm	498	496	0.41

Table 3.7 Comparison of Capacity by Lane (Based on Headways)

Time	Lane Capacity (pcphpl)		% difference Left Vs. Right
	Left	Right	
8.00 am – 10.00 am	357	372	4.20
11.30 am – 1.00 pm	462	497	7.04
4.00 pm – 5.00 pm	403	375	6.90
5.05 pm – 6.05 pm	522	504	3.45

The more conservative following behavior of small and medium-sized vehicles on the lanes largely explains why, as shown in Appendix B, the average time headway of all vehicles is larger than the average time headway of all small and middle-sized vehicles on the lanes. This following behavior, in turn, directly results in a reduction of the capacity of this particular lane. It should be noted that at the time of data collection, trucks were permitted to use lanes.

3.6 Effect Of Vehicle Size On The Capacity

The sample sizes by the headway types and lane for this one hour analysis period (5.05 pm – 6.05 pm) are shown in Table 3.7.

Table 3.8 Sample Sizes by Headway Types and Lane Distribution

Lane	Headway Type								
	S-S	S-M	S-L	M-S	M-M	M-L	L-S	L-M	L-L
Left	207	30	35	29	70	18	40	38	47
Right	190	34	40	25	39	20	31	36	39
Total	397	64	75	54	109	38	71	74	86

In order to compare the mean headways for different headway types, Duncan's multiple range tests (Walpole and Myers 1978) was applied. The results of the test are shown in Appendix C (Table C.1). It should be noted that the means with the same letter are not significantly different.

The results show that headways which involved small-sized vehicles with medium-sized vehicles, taken as a group, were significantly less than those which involved large-sized vehicles with either of the two or with each other.
From the results, it can be seen that the mean time headways adopted by large-sized vehicles following small or medium-sized vehicles are larger than the mean time headways in which large-sized vehicles are following large-sized vehicles.

From the above results it can be seen that large-sized vehicles drive more cautiously while following either small or medium-sized vehicles. During the peak hour of flow observed during this study period, the peak 15-minute interval occurred from 5:15-5:30. The basic value of capacity obtained by considering small-sized vehicles following small-sized vehicles during this peak 15-minute period is 575 pcphpl.

A study of the histograms frequencies of (as shown in Appendix C, Figures C.1 through C.9) reveals that on the whole the shape of the distribution is positively skewed (skewed to the right), differing only in the amount of skewness.

A particularly interesting observation which can be drawn from the skewness of the histograms is that when either small or medium-sized vehicles were the followers of each other the histograms exhibited a greater amount of skewness to the right than when large-sized vehicles were the followers of either small or medium-sized vehicles or of each other. This shows explicitly that for small and medium-sized vehicles, the high values of observations extend far above the mean and the low values are bunched close to it.

3.7 Estimation Of Passenger Car Equivalents

Passenger car equivalents (PCEs) would have to be known in order to convert a mixed traffic stream into an equivalent stream of passenger cars. The approach used is the one advocated by Krammes and Crowley (1987) who derived a formulation for PCEs beard on the proximity and freedom to maneuvers by using the mean lagging time headways for each combination of pairs of vehicle types that were found in the traffic stream.

By introducing the notations for headway types used in this study, the expression would have the following form:

$$PCE = [(1-p)(H_{L-S} + H_{S-L} - H_{S-S}) + p\,(H_{L-L})] \;/\; H_{S-S}$$

Where,
P = proportion of large vehicles in the mixed traffic stream.
H_{L-S} = average minimum time headway for large-sized vehicle followed by small-sized vehicle.
H_{S-L} = average minimum time headway for small-sized vehicle followed by large-sized vehicle.
H_{S-S} = average minimum time headway for small-sized vehicle followed by small-sized vehicle.
H_{L-L} = average minimum time headway for large-sized vehicle followed by large-sized vehicle.

The overall PCE values shown in the Table 3.8.

Table 3.9 Estimation of PCE values

Time	Vehicle Type	PCE values	
		Left	Right
4.00 pm - 5.00pm	Large	2.40	2.20
	Medium	1.09	1.03
5.05pm - 6.05pm	Large	2.66	2.46
	Medium	1.06	1.00

The results indicate that medium-sized vehicles do not affect the lane capacity and averaged value of PCE for the two time periods would be 1.00. The results also indicate that for large-sized vehicles the value of PCE, averaged for the two time periods, would be 2.43. It should be noted that the definition of 'large' in this study includes buses, trucks, trailers, etc.

4. CONCLUSIONS

The value of the basic capacity of interrupted flow highway facilities based on the average minimum time headway adopted by passenger cars following passenger cars during a peak 15-minute interval is about 575 passenger cars per hour per lane (pcphpl). Very close correspondence was observed between the computation of basic capacity based on the equivalent-hourly flow rate for the peak 15-minute of flow which gave a value of 498 pcphpl and that based on the mean time headway adopted by passenger cars in the same period. The difference between capacity values based on 5- and 15-minute flow rates averaged for both the peak and off-peak conditions was estimated at about 8.99%. During peak periods this difference was about 9.88% and during low-flow periods this difference was about 13%. The difference between capacity values based on 5- and 15-minute average minimum time headways averaged for peak and off-peak conditions was about 7.2%. During peak-flow periods this difference was about 5.80%' and during low-flow periods this difference was about 6.88%. An average speed of 40 km/h was obtained for the capacity flow based on the speeds of the passenger cars during the peak 15-minute of flow within the analysis hour. It was seen that the presence of large-sized vehicles had an adverse impact on the traffic stream. This impact was due not only to their large size and inferior operating capabilities but also due to the physical impact on nearby vehicles and the psychological impacts on the drivers of those vehicles which contributes to a reduction in capacity. It was observed that large-sized vehicles allowed more room to the front when following small and medium- sized vehicles than did drivers of small and medium-sized vehicles. The proportion of large-sized vehicles in the traffic lane is seen to affect the average headways of all types of vehicles. The average headways are seen to increase with an increase in the percentage of large-sized vehicles. This in turn led to a direct reduction of the capacity of the traffic lane. This reduction in capacity was also borne out by the results of the flow rates for peak 15-minute periods. The passenger car equivalents for large-sized vehicles appear as 2.43 and for medium size vehicles 1.00.

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REFERENCES

A Policy on Geometric Design of Highways and Streets (1984), American Association of State Highway and Transportation Officials, Washington, D.C.

Gwynn, D.W. (1968). “Truck Equivalency, Traffic Quarterly.” Vol. 23 (2), 225-236.

Highway Capacity Manual, Special Report 87 (1965), Transportation Research Board, Washington, D.C.

Highway Capacity Manual, Special Report 209 (1985), Transportation Research Board, Washington, D.C.

Krammes, R.A., and Crowley, K. W. (1987). “Passenger Car Equivalents for Trucks on Level Freeway Segments.” Transportation Research Record 109 Transportation Research Board, National Research Council, Washington, D.C., 10-16.

Roess, R. P., and McShane, W. R. (1987). “Capacity and Level-of-Service Concepts in the Highway Capacity Manual.” ITE Journal, Vol. 57(4), 27-30.

Walpole, R.E., and Myers, R.H. (1978). “Probability and Statistics for Engineers and Scientists.” Macmillian Publishing Co. Inc., New York.

APPENDIX A

EQUIVALENT HOURLY FLOW RATES

Table A.1 5 – min Flow Expressed in Equivalent Hourly Flow Rates

Time 8.00am -10.00am	Speed (km/veh)	Flow/5min (Veh)	Small	Medium	Large	PCPH
Left	60	0	0	0	0	0
	56	5	1	4	0	60
	50	4	0	4	0	48
	36	15	11	3	1	180
	35	29	15	11	3	348
	33	34	10	9	15	408
	32	31	17	10	4	372
	31	32	15	12	5	384
	30	33	18	6	9	396
	29	25	20	5	0	300
Right	60	0	0	0	0	0
	56	2	0	2	0	24
	50	0	0	0	0	0
	36	30	20	6	4	360
	35	36	14	10	12	432
	33	34	20	7	7	408
	32	33	15	10	8	396
	31	35	20	8	8	420
	30	32	26	6	0	384
	29	30	20	10	0	360

Table A.2 5 – min Flow Expressed in Equivalent Hourly Flow Rates

Time 11.30 am-1.00pm	Speed (km/veh)	Flow/5min (Veh)	Small	Medium	Large	PCPH
Left	40	24	18	3	3	288
	38	34	20	7	7	408
	35	41	26	11	4	492
	34	43	11	17	15	516
	33	42	24	10	8	504
	32	37	17	14	6	444
	31	40	22	5	13	480
	30	36	20	16	0	432
	29	28	10	15	3	336
	28	10	1	9	0	120
Right	41	20	0	17	3	240
	38	37	21	9	7	444
	36	34	16	13	5	408
	35	41	12	16	13	492
	34	32	14	8	10	384
	33	40	20	9	11	480
	32	38	21	5	12	456
	31	39	18	12	9	468
	30	35	11	17	7	420
	29	30	19	11	0	360

Table A.3 5 – min Flow Expressed in Equivalent Hourly Flow Rates

Time 4.00pm- 5.00pm	Speed (km/veh)	Flow/5min (Veh)	Small	Medium	Large	PCPH
Left	48	34	6	8	20	408
	42	24	5	9	10	288
	41	29	12	9	8	348
	40	30	12	8	10	360
	36	33	7	16	10	396
	35	35	17	8	10	420
	33	32	15	13	4	384
	33	28	6	20	4	336
	31	27	17	10	0	324
	30	25	13	12	0	300
Right	50	8	0	4	4	96
	46	10	2	4	4	120
	45	12	0	7	5	144
	40	22	8	12	2	264
	35	29	9	16	4	348
	34	30	16	8	6	360
	34	28	10	15	3	336
	33	25	11	12	2	300
	33	24	14	10	0	288
	31	26	16	10	0	312

Table A.4 5 – min Flow Expressed in Equivalent Hourly Flow Rates

Time 5.05pm - 6.05pm	Speed (km/veh)	Flow/5min (Veh)	Small	Medium	Large	PCPH
Left	45	10	1	4	5	120
	44	32	10	14	8	384
	40	30	15	11	4	360
	38	36	20	10	6	432
	37	40	12	17	11	480
	36	42	20	12	10	504
	35	46	19	15	12	552
	36	39	12	15	12	468
	33	35	18	10	7	420
	30	31	15	16	0	372
Right	45	15	23	20	2	180
	44	12	3	7	2	144
	38	35	17	5	13	420
	36	40	20	15	5	480
	35	38	13	5	20	456
	35	48	17	16	15	576
	34	39	15	20	4	468
	33	35	10	20	5	420
	31	33	10	17	3	396
	30	30	10	20	0	360

APPENDIX B

HEADWAY ANALYSIS

Values of Capacity have been recorded to the nearest vehicle by considering the reciprocal of the headways (before reading the headways to the significant decimal places as shown).

H = average minimum time headway for the headway type being considered

H' = mean time headway for the mixed traffic stream for the entire duration considered

Number = sample size of the headway type

Speed = average speed of vehicles involved in this type of headway interaction (km/h)

Table B.1 Headway Analysis of Period (8.00 am – 10.00 am)

Lane	Item	S-S	S-M	S-L	M-S	M-M	M-L	L-S	L-M	L-L	Duration (min)	Flow (veh)	H' (sec)	Capacity (veh/hr)	No. of Headways ≥ 7 sec
Left	Number	93	30	14	18	15	10	8	12	10	32	210	8.50	425	178
	H	8.78	9.81	9.38	7.60	7.51	9.78	8.41	7.88	7.00					
	Capacity	410	367	384	474	780	369	429	457	515					
	Speed	34.60	30.63	32.89	30.82	43.00	31.96	33.05	33.00	35.00					
	Number	30	22	12	7	10	4	6	8	15	15	114	10.10	357	80
	H	7.00	6.90	10.00	12.00	9.00	5.00	14.00	11.00	6.00					
	Capacity	515	522	360	300	400	720	258	328	600					
	Speed	36.25	32.00	31.00	30.70	36.00	33.00	30.00	30.60	30.00					
	Number	15	7	5	4	6	3	3	2	5	5	50	9.20	392	50
	H	6.00	9.00	10.80	10.00	9.00	7.70	9.50	10.50	7.30					
	Capacity	328	515	462	360	400	468	379	343	494					
	Speed	37.00	31.00	33.00	30.00	36.00	31.00	32.00	30.00	32.00					
Right	Number	81	25	10	16	13	10	7	14	11	32	187	8.80	410	180
	H	9.78	9.81	9.38	9.6	8.51	9.78	8.41	7.10	7.00					
	Capacity	369	367	384	375	423	369	429	508	515					
	Speed	32.00	30.21	33.2	32.22	36.00	32.79	30.63	32.69	31.00					
	Number	25	20	8	7	10	1	6	4	10	15	91	9.70	372	30
	H	5.00	6.50	5.70	6.80	8.00	10.00	7.90	15.00	12.00					
	Capacity	720	554	632	530	450	360	456	240	300					
	Speed	36.00	32.00	40.00	30.50	35.00	35.00	31.00	31.00	29.58					
	Number	8	7	2	4	6	3	3	1	2	5	36	8.40	429	21
	H	4.00	7.90	10.00	11.00	8.00	12.00	9.40	14.00	9.00					
	Capacity	360	456	900	328	450	300	383	258	400					
	Speed	37.00	32.00	42.00	31.00	34.00	30.00	30.00	29.00	32.00					

Table B.2 Headway Analysis of Period (11.30 am – 1.00 pm)

Lane	Item	S-S	S-M	S-L	M-S	M-M	M-L	L-S	L-M	L-L	Duration (min)	Flow (veh)	H' (sec)	Capacity (veh/hr)	No. of Headways ≥ 7 sec
Left	Number	201	70	50	28	31	16	22	30	32	60	480	7.50	480	230
	H	7.71	6.20	5.90	10.00	5.90	7.00	9.50	8.00	6.00					
	Capacity	467	581	611	360	611	515	379	450	360					
	Speed	33.91	30.76	32.53	32.76	36.71	30.55	34.13	33.03	35.70					
	Number	40	28	8	10	4	11	13	2	4	15	120	7.80	462	55
	H	5.71	8.00	6.00	10.00	5.90	8.00	9.00	8.00	5.00					
	Capacity	414	600	720	360	611	450	400	450	600					
	Speed	35.00	33.00	33.00	33.40	32.00	30.00	31.40	29.90	30.00					
	Number	15	14	1	5	2	2	2	2	0	5	43	7.40	487	30
	H	5.00	7.00	6.70	9.00	6.00	5.90	7.30	5.00						
	Capacity	600	515	537	400	600	1200	572	600						
	Speed	34.00	33.00	34.00	33.10	31.00	36.00	30.40	30.00						
Right	Number	208	67	52	28	37	18	22	28	30	60	490	7.35	490	290
	H	8.40	6.30	5.90	10.00	5.90	7.20	9.50	8.00	5.10					
	Capacity	429	572	611	360	611	500	379	450	706					
	Speed	32.11	31.26	32.50	32.70	37.71	30.55	32.13	33.00	36.70					
	Number	33	39	7	15	8	9	2	8	3	15	124	7.25	497	50
	H	4.00	9.00	5.00	10.00	7.50	9.20	6.00	6.60	8.00					
	Capacity	900	400	720	360	480	392	600	545	450					
	Speed	38.00	32.00	31.00	30.50	40.00	37.00	30.00	31.00	32.50					
	Number	15	7	3	3	5	1	3	2	2	5	41	6.48	554	15
	H	3.80	10.00	3.00	8.00	8.50	6.40	4.90	5.70	5.00					
	Capacity	948	360	1200	450	424	563	735	632	720					
	Speed	38.00	30.00	36.00	30.00	31.00	33.00	33.30	31.00	30.00					

Table B.3 Headway Analysis of Period (4.00 pm – 5.00 pm)

Lane	Item	S-S	S-M	S-L	M-S	M-M	M-L	L-S	L-M	L-L	Duration (min)	Flow (veh)	H' (sec)	Capacity (veh/hr)	No. of Headways ≥ 7 sec
Left	Number	188	40	22	30	34	14	27	12	28	60	395	9.11	395	220
	H	8.10	9.30	8.80	7.90	8.30	9.50	10.70	11.00	7.80					
	Capacity	706	319	462	303	494	379	308	343	530					
	Speed	33.00	32.00	31.60	32.30	36.10	31.50	31.00	34.00	35.50					
	Number	37	10	8	5	10	6	1	2	19	15	98	8.95	403	60
	H	6.00	9.00	11.00	8.00	7.00	10.00	7.90	7.60	12.00					
	Capacity	600	400	328	450	515	360	456	474	300					
	Speed	32.00	31.00	32.50	35.00	38.00	31.00	32.90	31.00	33.00					
	Number	13	2	5	1	3	2	2	1	6	5	35	8.80	410	33
	H	6.00	9.80	12.00	7.00	8.80	10.00	9.00	7.80	9.00					
	Capacity	450	368	300	515	410	360	515	462	400					
	Speed	32.00	33.00	31.00	32.00	36.00	31.50	32.00	32.00	33.50					
Right	Number	174	35	21	32	35	18	26	10	19	60	370	9.70	370	230
	H	7.50	7.60	12.00	7.90	6.50	10.70	12.40	13.00	8.30					
	Capacity	379	375	300	404	343	766	429	280	350					
	Speed	33.00	31.60	32.80	33.30	38.00	34.30	31.00	36.00	35.50					
	Number	26	3	16	9	10	1	5	2	20	15	92	9.60	375	55
	H	7.30	9.40	7.80	10.00	9.60	12.00	8.90	9.00	8.00					
	Capacity	372	383	462	360	474	300	405	400	450					
	Speed	33.00	31.00	34.00	31.50	35.00	33.00	32.00	31.20	30.00					
	Number	10	2	3	1	3	1	5	3	2	5	30	9.45	381	20
	H	7.00	9.40	13.00	8.90	10.50	6.00	8.00	14.00	10.30					
	Capacity	400	383	277	405	343	900	450	258	350					
	Speed	31.90	32.00	31.00	30.00	34.00	32.00	32.00	30.00	31.00					

Table B.4 Headway Analysis of Period (5.05 pm – 6.05 pm)

Lane	Item	S-S	S-M	S-L	M-S	M-M	M-L	L-S	L-M	L-L	Duration (min)	Flow (veh)	H' (sec)	Capacity (veh/hr)	No. of Headways ≥ 7 sec
Left	Number	207	30	35	29	70	18	40	38	47	60	514	7.00	514	345
	H	5.10	5.70	8.20	5.40	4.70	7.80	11.70	9.30	6.80					
	Capacity	500	632	440	383	766	362	360	680	706					
	Speed	34.70	33.00	31.00	35.00	40.00	38.00	34.60	36.00	37.40					
	Number	39	9	8	4	10	8	9	11	26	15	124	6.90	522	60
	H	5.00	6.40	7.30	8.00	5.80	7.80	9.00	8.90	6.60					
	Capacity	515	563	494	450	621	328	515	522	419					
	Speed	32.00	32.00	32.50	31.00	33.00	32.80	33.00	32.00	40.00					
	Number	19	7	2	2	6	2	3	1	4	5	46	6.30	572	29
	H	5.00	8.60	6.90	6.00	3.70	7.30	9.00	10.00	2.00					
	Capacity	600	419	522	720	973	494	400	360	1800					
	Speed	31.00	32.00	33.60	30.00	34.00	31.00	30.00	33.30	38.00					
Right	Number	190	34	40	25	39	20	31	36	39	60	454	7.93	454	230
	H	5.50	6.30	9.90	5.00	4.70	9.90	9.70	9.40	7.00					
	Capacity	480	494	405	360	538	522	468	429	450					
	Speed	33.00	33.40	31.00	35.00	36.00	33.00	32.20	30.00	35.40					
	Number	33	20	12	8	10	14	9	8	12	15	126	7.15	504	76
	H	5.00	6.80	7.70	10.00	7.90	8.00	10.00	8.00	6.00					
	Capacity	400	530	468	360	611	450	360	450	900					
	Speed	34.00	30.00	32.50	33.00	38.00	35.00	39.00	36.00	40.00					
	Number	15	10	4	5	2	3	2	2	5	5	48	6.40	563	22
	H	5.00	6.30	8.00	7.60	7.00	7.80	9.30	10.20	6.00					
	Capacity	515	572	450	643	515	462	388	353	720					
	Speed	32.00	31.00	30.00	32.00	42.00	36.00	34.00	30.00	38.00					

APPENDIX C

STATISTICAL ANALYSIS

Table C.1 Mean Time Headway by Headway Types

Grouping	Mean	N	Type
A	5.70	109	M – M
B	6.50	64	S – M
B	6.55	86	L – L
B C	6.85	74	L – M
D	7.35	397	S – S
D	7.35	38	M – L
E	8.55	75	S – L
E F	8.85	71	L – S
G	9.70	54	M – S

Note: Means with the same letter are not significantly different.

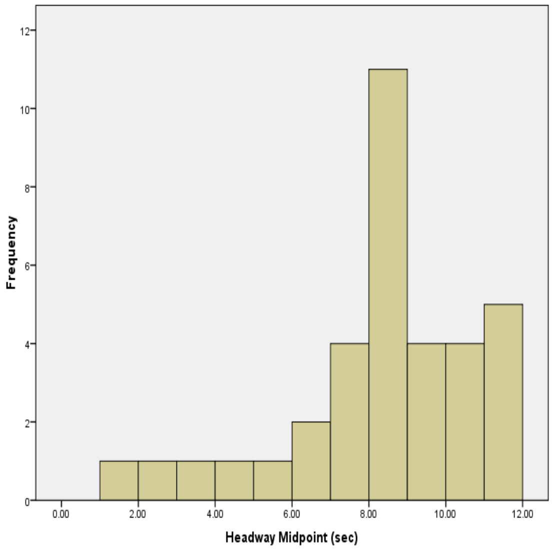

Figure C.1 Frequency Histogram S–L Headway Type.

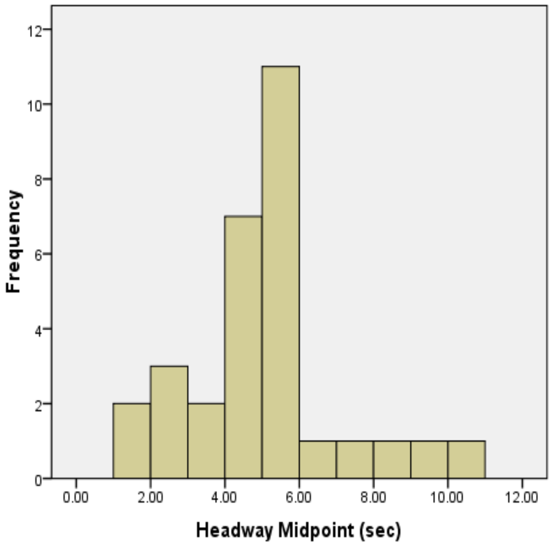

Figure C.2 Frequency Histogram S–M Headway Type.

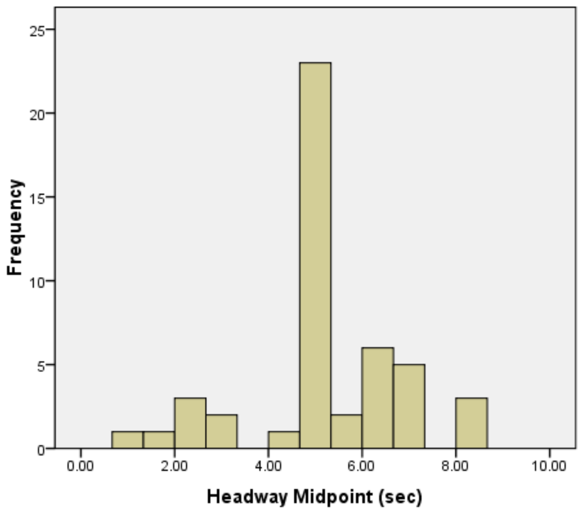

Figure C.3 Frequency Histogram L–L Headway Type.

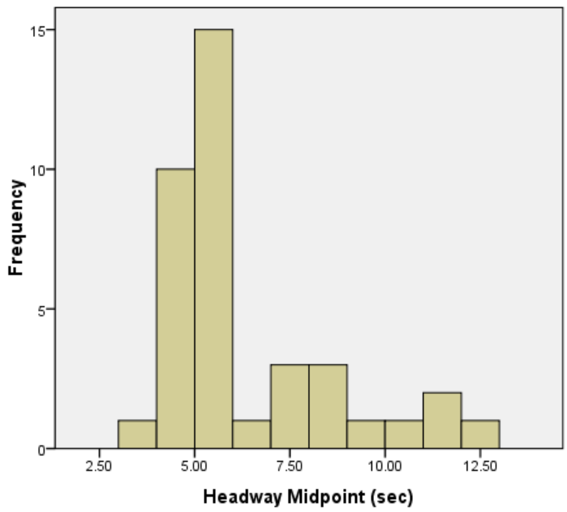

Figure C.4 Frequency Histogram L–M Headway Type.

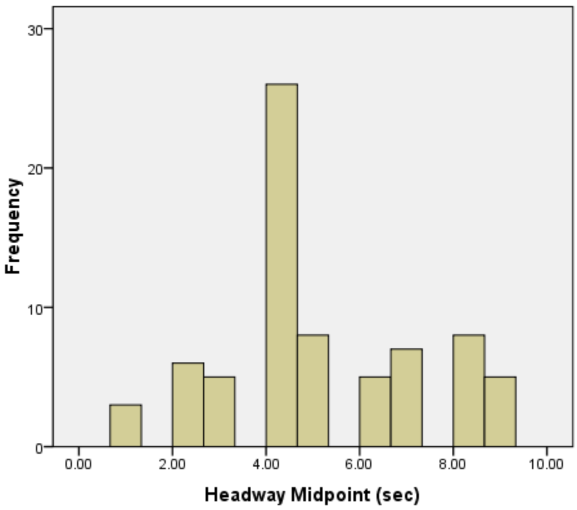

Figure C.5 Frequency Histogram M–M Headway Type.

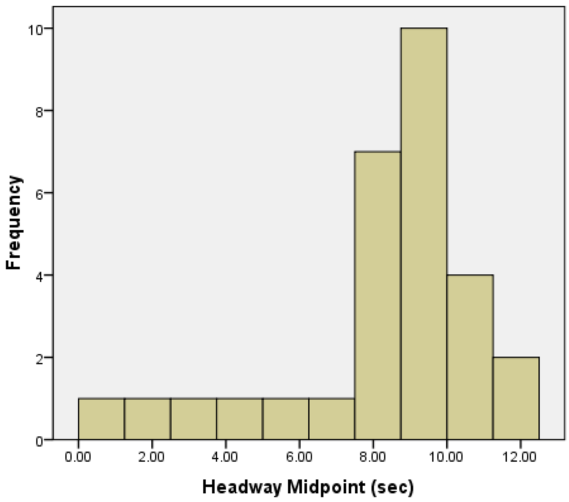

Figure C.6 Frequency Histogram M–S Headway Type.

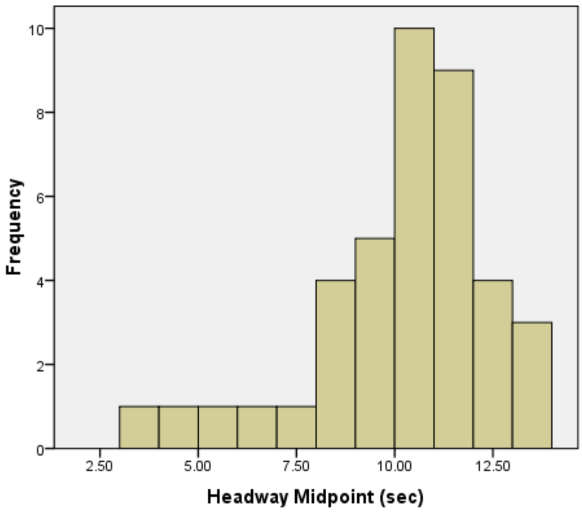

Figure C.7 Frequency Histogram L–S Headway Type.

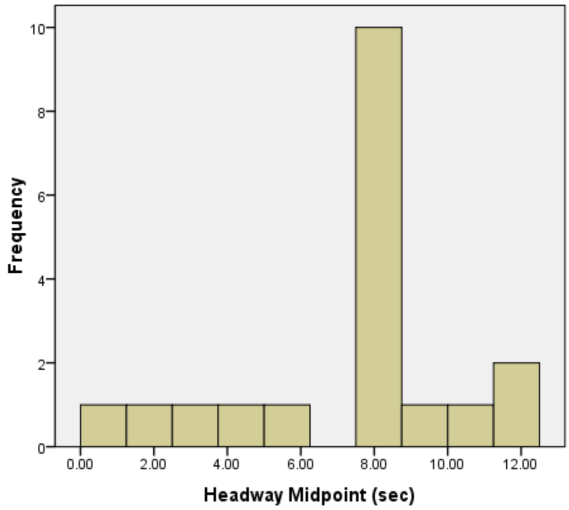

Figure C.8 Frequency Histogram M–L Headway Type.

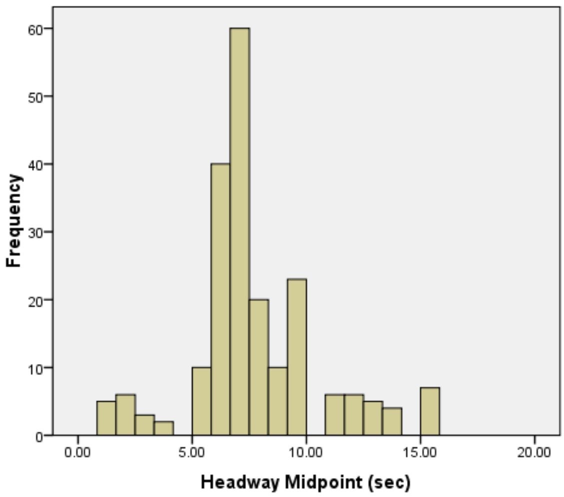

Figure C.9 Frequency Histogram S–S Headway Type.

A STUDY ON THE FACTORS INVOLVED IN TRUCK ACCIDENTS IN BANGLADESH

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ABSTRACT

The main purpose of this study is to identify the involvement of trucks in road traffic accidents in Bangladesh in addition to investigating the causes and to find remedial measures. Data on road traffic accidents in Bangladesh from 1998 to 2010 were collected using the Microcomputer Accident Analysis Package Five (MAAP5) software of Accident Research Institute (ARI). Field surveys were conducted at Gabtoli truck terminal and Dholaikhal. Trucks and heavy trucks are over involved in road traffic accidents and most of these accidents are fatal. Truck driver's activities contributed to 95 percent of truck accidents. Rest 5 percent were contributed by pedestrian and passengers, vehicular properties and road environment. From analysis, it is found that very few percentages of trucks involved in accidents were found with mechanical defects, physical damages and bad loading condition. But from field observations, damaged and overloaded trucks are seen everywhere and most of the uncovered trucks are found with very unstable loading. As young truck drivers are mostly involved in accidents, they should be made aware of the traffic rules and consequences of truck accidents. Traffic control systems, roadway delineations and rules on maintaining standard of body parts, weight and shape of loads should be improved.

Keywords: *Casualty, Overloading, Repair, Bangladesh, MAAP5.*

1. INTRODUCTION

Road traffic injuries constitute a major public health and development crisis of every country in the world. Nearly 3,500 people die on the world's roads every day, tens of millions of people are injured or disabled every year according to World Health Organization (WHO). These numbers will go on increasing with the increase of population as well as vehicle. Bangladesh has about 1.5 million motorized and could be over 3 million non-motorized vehicles (BRTA, 20011 – 2013). Each year, there are at least 3,000 fatalities and 3,000 grievous and simple injuries from around 3,500 police reported accidents on Bangladesh roads(Ahsan, H. M., 2012).

In Bangladesh, the involvement of truck and heavy truck in road accidents is significant. Trucks are known as the killer giants on highways. Almost 22 percentages of accidents had direct or indirect involvement of truck & heavy truck in the accidents occurred from 1998-2010 in Bangladesh. Up to June 2012, grand total of 95,922 truck vehicles are registered in Bangladesh according to BRTA. Truck and heavy truck accidents are increasing with this number. Moreover, percentage of fatal casualties for truck accidents has increased from 38 percent in 1998 to 56 percent in 2010 (MAAP5 analysis).

With the increasing number of freight vehicle usage, involvement of trucks in road traffic accidents will go on increasing if factors responsible for these accidents remain ignored and uncontrolled. So comprehensive studies on truck accidents will enable us to group reveal their contribution factors. Most of the times truck drivers' activities lead to hazardous accidents. Besides, pedestrians, passengers, road condition, vehicle properties and loading also cause accident with high casualty. The main purpose of this study is to identify the involvement of trucks in road traffic accidents in Bangladesh in addition to investigating the causes and to find remedial measures. This will lead to the identification of the problems involved in road traffic accidents and to plan strategies and countermeasures to improve the condition of road safety.

2. METHODOLOGY

This study is concerned about the truck accidents on the roads of Bangladesh in the period of 1998-2010. Extensive analysis and thorough field work was conducted to analyze the truck accidents with their variation with different parameters and reason behind over involvement of truck and their severity. Data were collected using the Microcomputer Accident Analysis Package Five (MAAP5) software of Accident Research Institute (ARI). Field surveys were conducted at Gabtoli truck terminal, and at Dholaikhal.

The major and minor contributing factors of truck accidents are analyzed in this paper. All the contributing factors are categorized in five distinct groups such as Driver's activity, pedestrians and passengers' fault, vehicle properties, road and roadside condition other factors. Further analysis on truck drivers' age and their casualties, road geometry, lighting and weather condition, traffic control on road, vehicle maneuver, defect, damage and loading are also done to find out factors behind truck accidents. Field surveys are mainly done to find out the exact loading condition of trucks and heavy trucks.

3. ACCIDENT DATA ANALYSIS

In the analysis of accidents, accident table shows the number of accidents varying with different types of factors such as year, locations, road classes etc. More than one vehicle can be involved in one accident but it will be count as one accident. Casualty table shows the number of persons injured or died in accidents. Vehicle tables show the number of a particular vehicle involved in road accidents.

3.1 Overall Accident and Truck Accident Trends in Bangladesh

The trend of road traffic accident for all types of vehicle and truck vehicle according to their severity is shown in the table below.

Table 1: Accident table: Accident severity from 1998-2010

Year	Fatal		Grievous		Simple		Collision		Total	
	All	Truck	All	Truck	All	Truck	All	Truck	All	Truck
1998	2611(52%)	726(63%)	1737	305	312	51	368	68	5028	1150
1999	3141(57%)	875(64%)	1485	303	493	102	400	88	5519	1368
2000	3289(59%)	856(63%)	1563	329	327	77	371	105	5550	1367
2001	2591(65%)	761(72%)	960	210	216	49	217	44	3984	1064
2002	3316(61%)	878(68%)	1316	254	313	78	447	78	5392	1288
2003	3591(63%)	913(70%)	1397	248	363	82	388	62	5739	1305
2004	3270(66%)	754(70%)	1031	198	340	72	303	52	4944	1076
2005	3086(69%)	745(73%)	937	190	211	41	235	40	4469	1016
2006	3507(72%)	792(76%)	889	164	191	45	277	39	4864	1040
2007	3785(70%)	765(72%)	1043	201	252	50	289	44	5369	1060
2008	3783(72%)	758(76%)	1023	161	236	38	245	36	5287	993
2009	2937(74%)	621(79%)	716	114	115	18	208	34	3976	787
2010	2612(76%)	506(80%)	585	92	110	13	143	18	3450	629
Total	41519(65%)	9950(70%)	14682	2769	3479	716	3891	708	63571	14143

Both the numbers of overall and truck accident have been reduced but percentage of fatal accident for both cases has been increased dreadfully throughout these years, from 52 percent to 76 percent for overall and from 63 percent and 80 percent for truck accidents with minor ups and downs in both cases. Although truck accident is 22 percent of all accidents, from this table it is clear that the fatality of truck accident is more dreadful compared to overall accidents.

Casualty data taken from the casualty table of MAAP5 software shows the number of persons injured or died in accidents. For truck vehicle, casualties of truck occupants such as drivers, passengers and helpers are shown in the figure below. From the average of these 13 years data, it is found that more than 46 percent of casualties caused to truck occupants are fatal. This percentage was only about 37 percent in 1998. It gradually increased to more than 45 percent in 2001. It reaches the peak in 2008-2010 (61-62%). In 2010 it is nearly 56 percent. It is seen that simple types of casualty is very rare case from 2004 indicating that truck accidents are getting more dreadful.

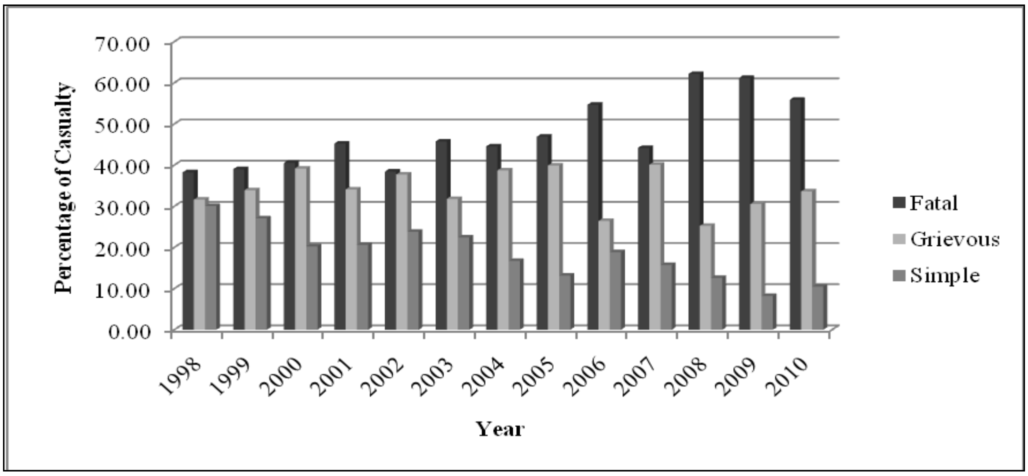

Figure 1: Variation of casualty of truck accidents (1998-2010)

3.2 Contributing Factors of Truck Accidents

Behind an accident, there may be one or more factors which are responsible for the accident. For improvement of accident scenario, the major factors need to be identified. For truck accidents the contributing factors are categorized in five distinct groups such as Driver's activity, pedestrians and passengers' fault, vehicle properties, road and roadside condition other factors which are shown in the table below

Table 3: Truck accidents contributing factors (1998-2010)

Major Factors		Fatal		Grievous		Simple		Collision		Total
		No	%	No	%	No	%	No	%	No
Drivers' Activity	Speed	5213	68	1595	21	403	5	496	6	7707
	Care	3996	73	1024	19	262	5	178	3	5460
	Sleep	30	81	4	11	3	8	0	0	37
	Close to vehicle	44	66	9	13	7	10	7	10	67
	Overturn	65	66	22	22	8	8	4	4	99
	Turning	20	65	6	19	5	16	0	0	31
	Alcohol	9	75	2	17	1	8	0	0	12
	Total	9377	70	2662	20	689	5	685	5	13413
Pedestrian and Passengers' Fault		257	88	33	12	3	1	0	0	293
Road and Roadside Environment	Signal	33	70	7	15	4	9	3	6	47
	Road Condition	23	70	6	18	0	0	4	12	33
	Road Feature	33	72	8	17	2	4	3	7	46
	Weather	15	71	4	19	1	5	1	5	21
	Total	104	71	25	17	7	5	11	7	147
Vehicle Properties	Vehicle Defects	32	80	7	18	0	0	1	3	40
	Load	20	74	6	22	1	4	0	0	27
	Tire Burst	8	80	1	10	0	0	1	10	10
	Total	60	78	14	18	1	1	2	3	77
Other Factors		41	73	9	16	5	9	1	2	56
Grand Total		9839	70	2743	20	705	5	699	5	13986

In 55 percent of the truck accidents, speed is the first contributing factor. Among all drivers' activities, sleeping caused maximum percentage of fatal accidents. Drivers' activities caused 95% of overall truck accidents. Other road users e.g. passenger and pedestrian were responsible for 2.1 percent, road and roadside environment contributed to 0.7 percent, vehicle properties were responsible for 1.1 percent of truck accidents. Among road and roadside environment factors, lack of traffic signal and road feature caused maximum number of accidents and their percentage of fatal accident is also high. Among vehicle properties, vehicle defects and tire burst caused most of the accidents. In case of overall truck accident, percentage of fatal accident is highest when passenger and pedestrian are involved. This is due to the fact that pedestrian is the most vulnerable and exposed among all road users.

3.3 Activities of Truck Drivers

Driver's casualty is an important factor to compare with the overall casualty to find out the main victim of truck accidents. It is found that the fatal casualties of truck drivers are very low (36.6%) compared to other victim's casualties. So there is a significant difference between driver's casualty and other's casualty. It is may be due to the reason that truck drivers always sit on a higher position protected by the steel body of truck. So they are less exposed and as a result the percentage of fatal casualty is always low.

For truck drivers, the age of maximum number of casualty is same as for all vehicles (31-35 years). The percentage of fatal casualties is also towards the elder drivers (61-65 years). Truck driver's age above 65 years is really rare because truck driving requires high body strength.

Maneuver of vehicle is a factor which has a great effect on the accident and it is entirely controlled by the driver. Number of truck vehicle analysis reveals that 79 percent of truck vehicles were involved in accident during "Going ahead" maneuver and 77 percent of accidents were fatal during "Sudden start" maneuver.

3.4 Road and Roadside Environment

Road and roadside environment includes junction type, lighting condition, road geometry, weather condition and traffic control. To identify the location of truck accident, junction type is another important factor. 75 percent of truck accident took place at mid block section of road. This may be due to the fact that highways have less number of junctions than other roads. Tee junction has the second most number of accidents which is 7.6 percent of total accidents.

The severity of accidents also varies with lighting condition. The figure below displays the variation of severity of truck accident with lighting condition.

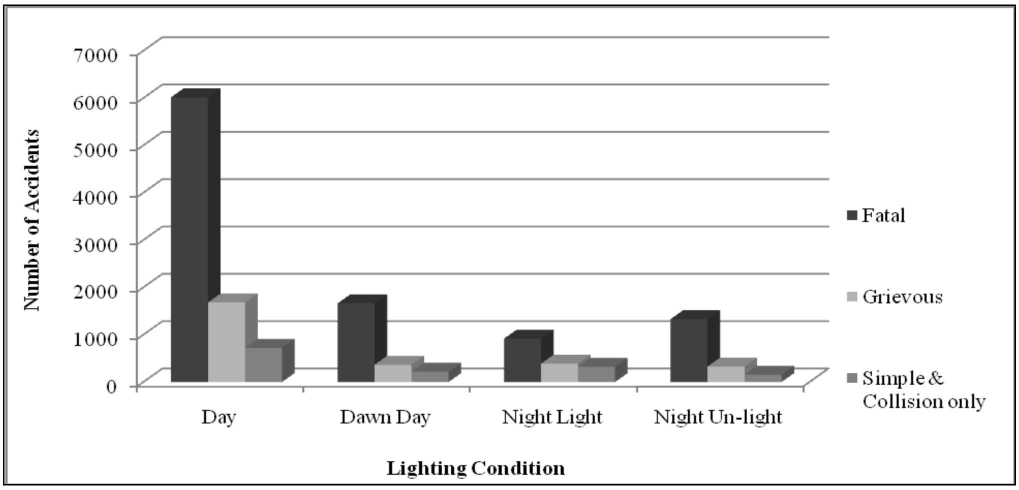

Figure 2: Truck accidents at different lighting condition

It is seen that in day light, number of truck accident is high. Almost 60 percent of all truck accidents occurred in day light and 71 percent of them are fatal accidents although Night un-light condition has greater percent of fatality (73%) which indicates that lack of lighting leads to fatal accident.

Geometry of road is an important road traffic factor, especially for accident analysis. It is found from analysis that, number of truck accident is very high (about 70%) in case of straight road. This may be due to the reason that most of the portion of a road is designed straight. Also speed of vehicle in straight road is usually high. But it was found that the percentage of fatal accident is highest in slope (75%) and lowest in crest (68%). This can be due to the fact that in slopes, vehicle speed is high and driver has less control on it and on crest, speed is lowest.

Trends of truck accidents in different weather conditions shows that maximum number of truck accidents occurred in fair weather condition (92.8%). Number of fatal accident is highest in case of foggy weather (73%). This is may be due to the fact that fog reduces site distance drastically fatal accidents are more likely to occur.

In Bangladesh, traffic control varies in urban and rural areas. In most urban areas, traffic is controlled by police or signal or both. In rural areas most of the roads are uncontrolled. Analysis of data shows that Most of the truck accidents occurred at "No control" situation (81.1%), where the drivers can maneuver as their wish. Also percentage of fatal accident is also high in this situation (72%). But fatal accident percentage is highest in case of pedestrian crossing (84%) and minimum where traffic is controlled by both signal and police (38%).

3.5 Truck Vehicle Properties

Trucks with defects in headlights, brake, steering wheel, tire etc. are seen everywhere. Besides, many of the trucks found on road are physically damaged. Loading conditions of them are not safe. But information on these factors is very difficult to report after an accident by the police as there are no established standards. To compare and to find deviation of police reporting from actual scenario, both data analysis and field observations were conducted on these factors.

3.5.1 Data on Vehicle Properties

From analysis of vehicle table data it is found that only 14 percent of trucks involved in accidents were found with mechanical defects. Among them, major defects are found in brake (15%), light (4%), tyre (3%) and other multiple defects. From figure below it is seen that although maximum number of trucks involved in accidents are with no defects, percentage of defected trucks have been increasing since 2003.

According to data, about 40 percent of trucks involved in accidents have physical damages. Major physical damages are frontal damage (51%), rear damage (8%), right side damage (4%), multiple damages (20 %) etc.

Loading of vehicle is an important factor to discuss the fatality of accident. Truck vehicle is mainly used for carrying freights. Overloading reduces the speed of truck but it increases the momentum. Maximum allowable load on truck is 7.5 tons and for heavy truck it is 22 tons. The following figure shows number of truck vehicle involved in road accident with their loading condition.

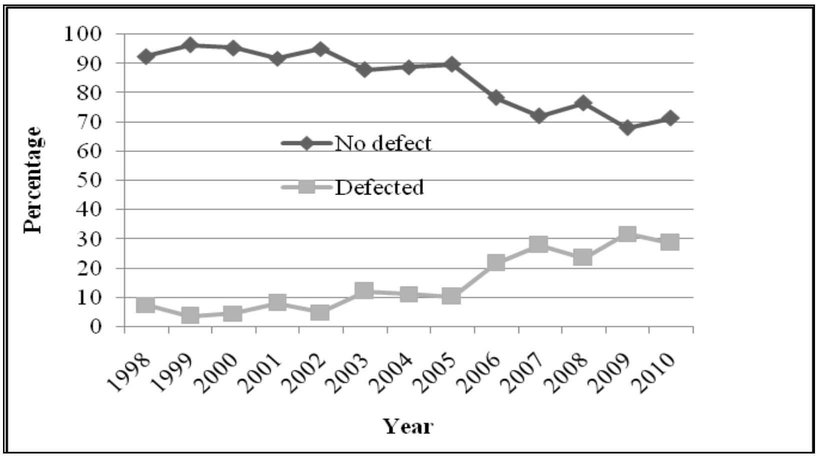

Figure 3: Variation of defected truck vehicle number

It is seen that bad loading has a little contribution to truck accident. In all of the years, weight of most of the trucks were within limit. But not only the weight but also the shape of loading has a great impact on accident severity. But this property is not listed in accident report form (ARF).

3.5.2 Field observations on vehicle properties

From observations on Gabtoli truck terminal and Dholaikhal it was found that most of the trucks carry load beyond their limit. Although it was not possible to measure the exact weight of the load, the size and shape of the loads were clearly hazardous. In many cases, the shape of truck loading is such that it crosses the side border of truck body. It varies for different types of loading, but it can be up to 1.5 feet. The figure below shows such a truck with extended loads. This type of loading increases both the possibility and severity of side collision with other vehicle. Collision with pedestrian is more likely due to this type of loading.

Excess height of loads also creates discomfort to other drivers. It is found that the height of the loads can be up to 10 feet. Due to this type of loading there creates a possibility of collision during passing under over bridge or similar structures

Figure 4: Side extensions of truck loads

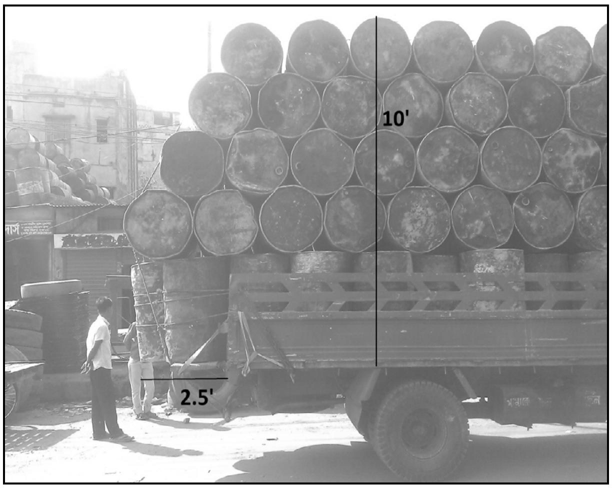

Figure 5: Height extensions of truck loads

Rear extension of loading is rare but it is really dangerous. The rear load is kept suspended by ropes or by other means. In many cases the total portion of the load is out of the truck body. This type of loading increases the possibility of rear collision and makes discomfort to other vehicle drivers.

The system by which loads on truck are kept stable is mainly made of ropes anchored in hooks of the truck body. From figure below, it is seen that the side board of truck body is held just by a rope which can be displaced from the hook any time. Then the load will lose support and may create hazardous accident. Trucks were also found carrying unstable loads. These types of loads should be carried by covered truck rather than open carrier truck.

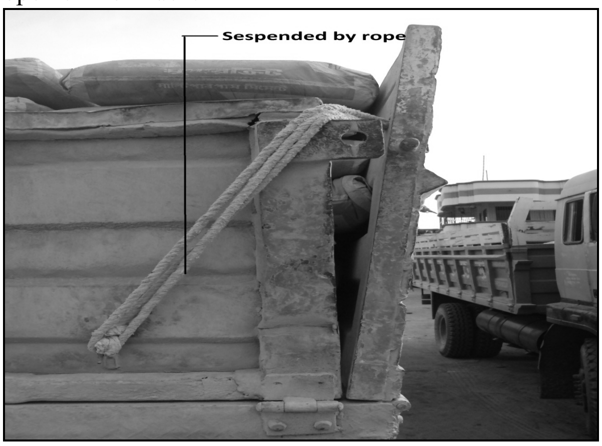

Figure 6: Truck body suspended by ropes

Figure 7: Uncovered truck carrying unstable loads

From field observations, damaged and defected trucks were found being repaired by local workers. According to the opinions of these workers, owners of trucks are more interested in repairing rather than buying new trucks. It is seen from the figure that the broken parts of a truck which were involved in an accident are being jointed by welding. Missing parts of the truck body are replaced by locally available body parts according to the worker. Thus this truck will again move on roads. Even the tires of trucks were found being repaired by unskilled workers

Figure 8: Repairing of truck body

Figure 9: Sewing of damaged tires

4. CONCLUSIONS

Truck accidents are getting more severe day by day because of lack of awareness and laxity of law. Lack of driver's control, improper design of highway, use of defected vehicle and vehicle parts, oversized loading are the main issues that putting the severity of truck accident on an increasing trend. The following countermeasures can be suggested to reduce truck accident and accident severity in Bangladesh

- As young truck drivers are mostly involved in accident, they should be made aware of the traffic rules and of the vigorous consequence of truck accidents. Over aged drivers must be retired and provided with elderly allowance
- Truck owners should be aware of duration of driving and resulting fatigue of each driver.
- Traffic control systems well as road delineations should be improved. In highways, speed of truck has to be controlled by imposing speed limit. At hazardous location highway police should keep an eye to identify who is crossing speed
- Rules should be made on maintaining standard of body parts, weight and shape of carrying goods and they should be imposed properly

REFERENCES

Ahsan, H. M. (2012). "Road Safety in Bangladesh: Key issues and countermeasures".ARI Road Traffic Accident Database.

BRTA, Ministry of communications, Government of the People's Republic of Bangladesh, "National road safety strategic action plan 20011 – 2013".

EVALUATING THE PERFORMANCE OF A ROAD: A CASE STUDY OF MURADPUR TO DEWANHAT ROAD, CHITTAGONG

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ABSTRACT

Efficiency of transportation network has greatly influenced by traffic volume and capacity, speed, geometrical design, road side land use etc. Besides, all these factors have a great impact on the overall performance of the road. The aim of this research is to find out the performance of the feeder road connecting from Muradpur and Dewanhat junction in Chittagong City, second largest city of Bangladesh. The study road is very important road of the city because it is connected with Bahaddarhat, GEC Circle, WASA circle, Tigerpass, Dewanhat and many other important places of the city boundary and these areas have a great economic impact in the entire city. To evaluate the performance of this road, its geometrical characteristics and Level of Services (LOS) have been analyzed depending on the data collected through different field surveys. The present condition of the study road was tried to be focused by making a comparison between the measured data from survey and national and international standard. This research also explains how road way and traffic factors relating to LOS greatly influence the performance of the road. It was found from various surveys that the performance of this road was completely failed to meet the traffic demand during peak hour and traffic congestion is reached in an intolerable position. The research helps us to understand the magnitude of different factors to maintain a good performance of a road.

Keywords: *LOS, Speed, Volume, Road factors, Performance*

1. INTRODUCTION

1.1 Background information

Bangladesh is one of the most densely populated countries in the world bearing a population of 965 per square km (BBS, 2011). Rapid urbanization has also become a visible feature, especially in the three major cities of Bangladesh: Dhaka, Chittagong & Khulna, after the liberation of Bangladesh in 1971. While the total population of the country has been increasing at 1.37% per annum (BBS, 2011), the urban population is growing at about 3.27% per annum (Aqua Consultant et al, 2012). This rapid urbanization has created a strain on the resources of local bodies like cities, towns and municipalities, which are often finding it difficult to cope with the increasing demands of the city dwellers for urban services and civic amenities like transport, housing, utility services. Like all developing countries transportation is a very important ingredient to their economic activities (Mahmud et al, n.d.).To cope with the rapid urbanization and diversification of economic activities as a developing country the transportation system of Bangladesh is facing numerous problems such as extreme degree of traffic congestion (Hodgkin son and Ellery, n.d.). Like other countries, the traffic problems are increasing day by day and have become a burning issue for Bangladesh (Shamsher and Abdullah, 2013). In Bangladesh the overall annual growth rate has been nearly 8.2 percent for fright transport and 8.4 percent for passenger transport (Mahmud et al, n.d.). Annually country wise economic wastage caused by traffic jam was USD 79 million for the year 1997. The traffic congestion cost is USD 03 billion a year and the city losses over 08 million work hours daily (Shamsher and Abdullah,2013).Chittagong is the second largest city in Bangladesh having a population of about five million including people living in the urban fringes. The functional operation of Chittagong city is largely depends on the existing road pattern into city area. The concept, 'functional operation' has been brought out from the concept of 'Transport Efficiency' (Debnath and Islam, 2009). Because of increasing population in this city, the efficiency of transportation system is becoming more and more complicated as like Dhaka city. This poor traffic system is being stimulated by heavy concentration of traffics, absence of adequate public transport, inadequate road infrastructure, faulty signaling equipment and poor enforcement of traffic rules (Shamsher and Abdullah, 2013). In order to keep the wheels of development, moving a sound and well performing transport system is must.

1.2 Statement of the problem

Chittagong city is experiencing rapid growth of population because of rural urban migration (1997; BBS 1981, 1991 and 2001). This rapid growth of population increasing the traffic pressure on road and making city dwellers life standstill due to traffic congestion (Shamsher and Abdullah, 2013).Chittagong city is functionally connected with surrounding area by mainly one road named Kalurghat to Patenga. Intolerable traffic jams as well as low performance at this important road especially in the Muradpur to Dewanhat segment have made lives of citizen miserable. The traffic system of this segment has already collapsed as the vehicles can't run on the main road of the city due to excessive congestion. According to experts opinion traffic congestion occurred in Muradpur to Dewanhat road mainly due to huge traffic flows and mixture of motorized and non-motorized vehicles ply at the same time in the main road intersections. As per BRTA (Bangladesh Road Transport Authority, 2010) source at present the total number of approved vehicle in Chittagong city is 84391. Among these, Bus -2816, Auto rickshaw (Taxi)-20847, Tempo-4666, Motorcycle-13470, Jeep-1951, Car-15961, Pickup-3656, Tank lorry-386, Tractor trailer-503, Truck-14065 and Micro-5998 where city roads are not enough to capable of carrying all this type of traffic especially in the peak hours. Day by day all these vehicles are gradually increasing. The road between Muradpur to Dewanhat segment is not well wide spread and getting narrower because of illegal possession on the road. According to DAP, 2008 road width of this road has recommended as 100 to 120 feet. But most part of this road have approximately 80-90 feet black top only. Illegal on street parking, high volume of traffic, poor lane management, improper planning of this road is decreasing its performance day by day.

1.3 Aims and objectives

Analyzing the performance of Muradpur to Dewanhat road in Chittagong city is the aim of this study. The road was sub divided into three segment named Muradpur to 2 no gate link, 2 no gate to GEC link, GEC to Tigerpass link and for fulfilling the aim traffic volume, speed, capacity and level of service for each link were calculated. Considering the factors that were affecting the performance of study area is also one of the key objectives of this research.

2. STUDY AREA

For analyzing performance and issues relating to the performance of a road, we have taken Muradpur to Dewanhat road of Chittagong city as our study area. This road contains major three intersections: 2 no gate, GEC and Tigerpass. Two very important roads named Chawkbazar and Hathazari road is connected with 2 gate junction whereas Dhaka-Chittagong highway is connected with GEC and New market, Pahartali and Agrabad road is connected with Tigerpass intersection point. This road is the major feeder road of Chittagong City Corporation and it provides numerous commercial, utilities and service centers of its both sides. Therefore most of the time it remains very working occupying different mode of vehicles by users.

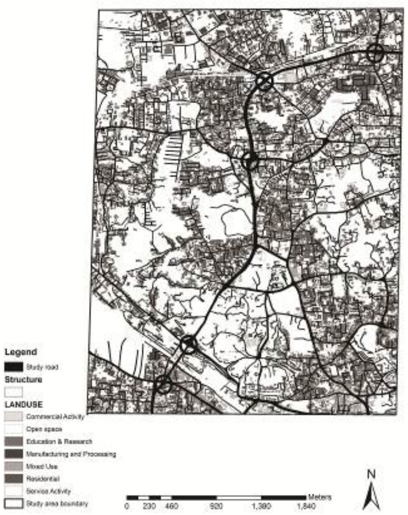

Figure 1: Study area

3.

4. METHODOLOGY

The research consists of both primary and secondary data. Primary data were collected from reconnaissance survey and field survey. Secondary data were collected from literature review, newspaper reports, websites etc. The geometric data of the study road were collected through field survey and for collecting finding Level of Services (LOS) of the road some data relating to volume, speed etc. were collected through field survey and some data were extracting through different formulas. In this research for finding the LOS of the study road, volume and capacity ratio is used. The study road was selected from Muradpur to Dewanhat junction. To find the volume data we have counted traffics from three major intersection points (2 No. Gate intersection, GEC intersection and Tiger pass intersection) in different direction using manual method both in holiday and working day. In this method we personally counted and classified traffic flowing. Then the counted value of the traffic was multiplied by the relating PCU value of the traffic. By the way the volume data was found .We divided our counting time into peak and off peak hour. The time durations were 8:30-9:30 am, 11:00-12:00 am, 03:00-04:00 pm and 05:30-06:30 pm. The vehicular types and their relating PCU are given in the chart:

Table 1: Vehicle type with PCU

Vehicle type	PCU
Bus	2.5
Minibus/Truck	2
Car/Microbus/Zeep	1
CNG	0.5
Rickshaw	0.8
Tempo/Human hauler	0.6
Motorcycle	0.3
Bicycle	0.2
Push Chart	4

Source: Roads and Highway department of Bangladesh

For calculating the capacity the following theoretical formula was used:
Here flow rate (q) is obtained by moving observer method which was conducted at the same time of volume count. In this method flow was measured into selected directions. The using formula was use to find the flow rate:

$$q_{a-b} = \frac{X_{b-a}+Y_{a-b}}{t_{a-b}+t_{b-a}} \quad \text{......} (1)$$

q_{a-b}= flow rate to b direction, X_{b-a}= opposing traffic count of vehicles met when the test vehicle was travelling froma to b, t_{a-b}= journey time of a to b, t_{b-a}= journey time of b to a, Y_{a-b}= number of vehicles overtaking the test vehicle – the number of overtaken by the test car, when the test car was travelling b direction.
For calculating average journey time the following formula was used:

$$\bar{t} = t - \frac{y}{q} \quad \text{......} (2)$$

t= total journey time, q= flow rate, y= no. of overtaking vehicles- no. of overtaken vehicles when test vehicles was moving
For calculating average journey speed (v) we used following formula:

$$V = \frac{d}{\bar{t}} \quad \text{......} (3)$$

d= total distance, $\bar{t}$= average journey time
Again for calculating average spacing (S) we used following formula:

$$S = L + .278vt \quad \text{......} (4)$$

V= average journey speed, L= length of the test vehicle in meter, t= perception-reaction time in second. In our research L= 2.63 meter and t= .75 second

$$C = \frac{1000V}{S} \quad \text{......} (5)$$

C= Capacity in vehicles per hour, V= average journey speed in K.P.H, S= Average spacing in meters of moving vehicles.
After calculating the LOS, it was justified through geometrical characteristics of study road, road way factors and traffic factors.

5. RESULTS

5.1 LOS analysis

Firstly the volume data was collected using the method mentioned earlier from three major intersections of the study road for definite direction. Then from speed survey average journey time was measured in the way which was mentioned in methodology part. Once volume and speed survey was completed, average journey speed was calculated from dividing total length of the study road by average journey time. Average spacing for study road was calculated from considering the perception time braking distance and length of the vehicle by using the formula stated in methodology.

Table 2: Link to link information for working day (Volume survey and speed survey)

Link Name	Average journey time (minute)	Average journey speed (Kmh^{-1})	Average spacing (meter)	Capacity (PCU/hr.)	Volume in working day (PCU/hr.)
Muradpur to 2 No Gate	8	8.25	4.35	1896.55	1401.85
2 No Gate to Muradpur	23.48	2.81	3.21	1875.38	1855.25
2 No. Gate to GEC	6.11	7.85	4.26	1842.72	1685.07
GEC to 2 No. Gate	13.71	3.5	1.91	1832.46	2051.3
GEC to Dewanhat	8.56	16.12	6	2686.66	2295
Dewanhat to GEC	8.78	15.72	5.91	2659.89	2587

Source: Field survey, 2012

Table 3: Link to link information for holiday (Volume survey and speed survey)

Link Name	Average journey time (minute)	Average journey speed (Kmh^{-1})	Average spacing (meter)	Capacity (PCU/hr.)	Volume in holiday (PCU/hr.)
Muradpur to 2 No Gate	7.67	8.6	4.42	1945.70	1194.55
2No Gate to Muradpur	8.27	7.98	4.29	1890.14	1417.45
2 No. Gate to GEC	6.41	7.48	4.15	1785.20	1447.45
GEC to 2 No. Gate	6.89	6.96	4.08	1705.88	1723.7
GEC to Dewanhat	9.29	14.84	5.72	2594.41	1801.9
Dewanhat to GEC	9.87	13.98	5.54	2523.46	1846

Source: Field survey, 2012

Table 4: LOS according to V/C Ratio for different links

Link Name	Working Day		Holiday		Standard	
	V/C Ratio	LOS	V/C Ratio	LOS	V/C Ratio	LOS
Muradpur to 2 No Gate	0.74	C	0.61	C	0	A
2 No Gate to Muradpur	0.99	E	0.75	C	0.0 - .12	B

2 No. Gate to GEC	0.91	D	0.81	D	0.12- .75	C
GEC to 2 No. Gate	1.12	F	1.01	F	0.75- .95	D
GEC to Dewanhat	0.85	D	0.69	C	.95- 1.0	E
Dewanhat to GEC	0.97	E	0.73	C	>1	F

Source: Analysis by authors, 2012 and High way capacity manual (TRB), 2000

From the above table it is shown that at working day among the six directional movements of vehicles one segment's (GEC to 2 No. Gate) LOS is F. Only in Muradpur to 2 No Gate it is C and rests of them it varies from D to E. In case of holiday there is no change in LOS for GEC to 2 No. Gate and in other segment it is C. So it can be concluded that during both holiday and working day GEC to 2 No. Gate intersection point is completely unable to meet traffic demand whereas other intersection points are at vulnerable condition. As a result congestion in our study area is very much common scenario.

5.2 4.2 Geometric Design Analysis

Cross sectional elements embrace aspect such as right of way width, road way width, pavement width, median, shoulder, and clearances. Muradpur to Dewanhat road is a feeder road considering its carriage way consists of 2 lanes. How calculated LOS was justified through geometrical characteristics of our study road is shown through the comparison between geometric design standard according to the Roads and Highway Department of Bangladesh and existing characteristics of cross sectional elements in study area:

Table 5: Comparison between geometric design standard and existing characteristics of cross sectional elements

Name of cross section	**Right of way (feet)**	**Standard(feet)**	**Road way width(feet)**	**Standard(feet)**	**Carriage way width(feet)**	**Standard(feet)**	**Central reservation(feet)**	**Standard(feet)**	**Shoulder (feet)**	**Standard(feet)**	**Curbs (feet)**	**Standard(feet)**	**Clearance (feet)**	**Standard(feet)**
2 no gate to Muradpur	98.8	---	42.9 & 34.3	71	42.9 & 34.3	48	10	--	----	6	3	1.44	--	3.28
2 no gate to GEC	128	---	57.3 & 43.3	71	57.3 & 43.3	48	10	--	----	6	2.3	1.44	--	3.28
GEC to 2 no gate	118.9	---	45.1& 48.8	71	45.1 & 48.8	48	4	--	----	6	1.5 & 1	1.44	--	3.28
GEC to Dewan hat	112.9	---	44.1 & 47.7	71	32.10 & 34	48	4.4	--	12 & 13.7	6	1	1.44	--	3.28
Dewan hat to GEC	76.6	---	32.1 & 29.1	71	22.1 & 21.7	48	1.9	--	10 & 7.4	6	1.1	1.44	--	3.28

Source: Field survey, 2012 and Geometric design standard for Roads and Highway department of Bangladesh, 2000

The above table states that existing road way width and carriage way width both are same and below standard value, meaning part of shoulder and clearance are absent in study road. As a result on street illegal parking is happened spontaneously on carriage way part and creates heavy traffic congestion. Besides during peak hour the study road is unable to occupied large volume of traffic due to inadequate carriage way space. In case of 2 No. gate to GEC segment, carriage width is almost equal more than standard value but width of road way is below standard, meaning that due to absence of shoulder this segment is completely unable to occupy the pressure of vehicles during both working day and holiday as this. Thus its LOS was F. Same cause is responsible for bad LOS at GEC to 2 No gate intersection in working day but during holiday it seemed better than working day. On the other hand GEC to Dewan hat and Dewan hat to GEC both segment has proper shoulder but due to too narrow carriage way width made the road inefficient during working day. In case of 2 No gate to Muradpur segments' LOS was bad both in holiday and working day due to insufficient width of road way and carriage way.

5.3 4.3 Factors affecting level of service

The factors which affect the level of service of study road can be considered under the following categories:

Lane width: A lane width of 3.65m is considered as the defined ideal lane (Kadiyali, 2003). In Muradpur to Dewanhat road, average lane width is 3.67m. But there are differences in lane width in different sections of the Muradpur to Dewanhat road. So difference in lane width and number of lane affects the level of service.

Shoulders: Shoulders of adequate width are necessary for maintaining traffic continuously. The highway capacity manual reckons that for lanes less than 3.65m wide, paved shoulders of 1-2 m or more width increase the effective width of the adjacent traffic length by 0.3m. There is no shoulder in Tiger pass and 2 No gate intersection but GEC intersection has shoulder.

Surface condition: A deteriorated and poorly maintained pavement adversely affects level of service. The surface condition of on Muradpur to Dewanhat road is different in different places. At 2 no gate the surface condition is not at satisfactory level due to construction work and GEC and Tiger pass intersection road condition are well enough.so surface condition affects the level of service.

Channelization: Traffic channelization is very important to control the flow of traffic. With thehelp of proper channelization the flow of traffic increase at high rate and traffic accident reduces. All the intersection Tiger pass, GEC and 2 No gate have medians, road dividers, Roundabout, traffic islands etc. proper Channelization greatly affects level of service.

Land use: As maximum land beside our study area is used as residential and commercial purposes. As a result people of this area are always working with their own work and thus congestion of people in this study area is very high and creates numerous obstacles to vehicles especially for drivers. So the general performance of this road is obstructed greatly. Following figure shows the general land use pattern in study area:

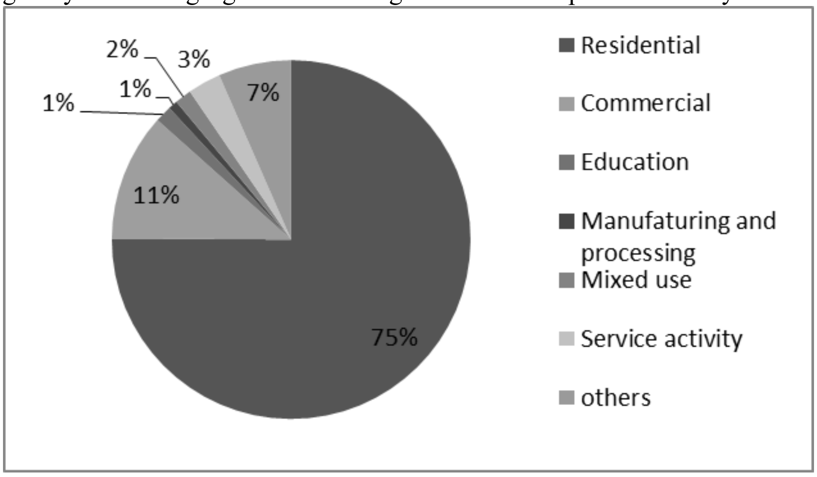

Figure 2: General land use pattern in study area
Source: CDA, 2007

Parking facility: It is one of the major problem in our study area because of illegal on street parking system creates lots of traffic congestion as well as cause of low speed of vehicles. Due to low parking facility, people park their vehicles at the road side which affect the level of service.

Existing traffic signal effect: Traffic signal system in our study area is very much poor because of improper traffic management. During survey it was found that there is no provision of digital traffic signal system as a result manual traffic system creates lots of problem like congestion, accident which effects largely on performance of this road.

On street parking: The road side shopping malls have no parking space though some malls have but it is not enough. For this reason it was found so much on street parking. Besides other factors are influencing on street parking like behavior of the drivers to park the vehicles on the street.

Small Motorized Vehicles: From Muradpur to Dewanhat road there have been found a huge number of small motorized vehicles having small Passenger Car Unit (PCU). This large volume of small vehicles are occupied most of the space of our study road and creates intolerable congestion.

Concentration of stoppage: The total length of study area is 4.5455 km. between these small lengths there are five stoppages which are in Muradpur, GEC, WASA, Lalkhan bazaar and Tigerpass. As a result during peak hour congestion becomes very high. So the increase number of stoppage affects the level of service. But in USA the minimum distance one stoppage to another stoppage is 5 km. while in UK it is 3 km.

Frictional effect: Frictional effect in our study road is not so severe though there are some places were found during survey where road condition seems to be affected by maintenance work done by utility department and also due to heavy load. As a result speed is being hampered.

Bus Bays and Stopping Places: It is completely absent in our study road as a result bus stops haphazardly and creates lots of trouble to other vehicles and road users

Drainage: Due to bad drainage system in our study road, during rainy season water stands on road and affect largely on level of service.

Footpath: It is one of the main causes of affecting the level of service of Muradpur to Dewanhat road. During survey poor maintenance of footpath was found in different parts of the road. Surface condition is not at satisfactory level and the path way is not wide enough. Most of the space of footpath is occupied by hawker and street dustbin. As a result pedestrians are forced to walk on main road.

6. CONCLUSIONS

There are many ways to measure transportation system performance, each reflecting particular perspectives concerning who, what, where, how, when and why. Different methods favor different types of transport users and modes, different land use patterns, and different solutions to transport problems. The Levels of Service is one of the most important factors in highway design. Highway design engineers and planners have to follow the levels of service criteria to determine the number of lanes and other geometry to suit with predicted traffic volume for ensuring better performance of road. The purpose of this study is to find out the level of service (LOS) and the factors affecting the LOS of a feeder road within the limit of Muradpur to Dewanhat junction. The Level of Service (LOS) is a composite of several operating characteristics that are supposed to measure the quality of service as perceived by the road users at different flow levels. Different types of practical observation methods and secondary sources were used to collect the required data. This study is helpful to identify the existing problem of Muradpur to Dewanhat road as well as its functional characteristics.

7. REFERENCES

Mahmud, S.M.S., Rahman, M.W. and Rabbi, S.H.E. (n.d). "Transport System in Bangladesh: Issues and Options for Sustainable Development", http://www.buet.ac.bd/ari/

Hodgkinson, D. and Ellery, S.W., (n.d). "Strengthening Bangladesh: Transport, Sustainability and Better lives", http://www.hodgkinsongroup.com/documents/transportation_sustainability.pdf.

Shamsher, R. And Abdullah, M.N., (2013), "Traffic Congestion in Bangladesh- Causes and Solutions: A study of Chittagong Metropolitan City",http://abrjournal.weebly.com/uploads/1/1/3/2/11327577/3.2_abr_template.pdf.

Debnath, R and Islam, M.S., (2009). "Managing efficiency of an urban road toward better transport system", http://academicjournals.org/journal/JGRP/article-full-text-pdf/16399851077

Kadiyali, L.R., (2003). Traffic Engineering & Transportation Planning, New Delhi, India. Mathew.V.T, Fundamental Parameters of Traffic Flow, Traffic Engineering and Management, Institute of Information and Technology, Bombay.

Bangladsh Road Transport Authority, 2010. http://www.brta.gov.bd/

Roads and Highway department of Bangladesh, (n.d). http://www.rhd.gov.bd/.

Transport Research Board, (2000). Highway Capacity Manual for Bangladesh.

Bangladesh Bureau of Statistics. Population Census 1981, Community Series, Zilla: Chittagong.

Bangladesh Bureau of Statistics. Population Census 1991, Community Series, Zilla: Chittagong.

Bangladesh Bureau of Statistics. Population Census 2001, Community Series, Zilla: Chittagong.(Dhaka:BBS-2007).

Aqua Consultant et al (2012). Study on Municipal Solid Waste Management. Professional Report. Bangladesh Municipal Development Fund.

Chittagong Development Authority, (2008). Detailed Area Plan (DAP) for Chittagong. (Chittagong:CDA-2008)

Roads and Railway Division, (2000). "Geometric Design standards for roads and Highway department, Draft version-4" Government of the People's Republic of Bangladesh, Ministry of Communications

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IRAP STAR RATINGS FOR NATIONAL HIGHWAYS IN BANGLADESH

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ABSTRACT

The alarming rise of traffic crashes is immensely affecting the social and economic development of Bangladesh. Every year 20,000 people are killed on Bangladesh roads and more than 40% of the crashes occur on National Highways, although they comprise of 16.5% of the major road network. Road infrastructure is considered as one of the pivotal contributing factors for most of the serious crashes and therefore aiming towards adopting safe design by improving road infrastructure could improve the situation. Road infrastructure based assessment programme has been proved to be successful in developed countries and now it is time to use this proactive approach in developing countries like Bangladesh. iRAP, the International Road Assessment Programme, works around the world on reducing the hazardous roads which retain the risk of road crashes that cause deaths and severe injuries and estimates the level of risks of the roads for safety. It also features countermeasures in terms of affordable road infrastructure improvements that can reduce the likelihood of crashes. This paper describes the iRAP methodology of star ratings used in recent iRAP-ADB Road Safety Improvement Programme in Bangladesh and presents some of the striking features of the assessment. It also emphasizes the opportunities of reducing the number of fatalities in the high risk corridors to fulfill the road safety targets of Bangladesh during the Decade of Action.

Keywords: *National Highway, Road infrastructure, iRAP, Star Ratings, Risk Assessment*

1. INTRODUCTION

Bangladesh is going through enormous road safety crisis which should be dealt with more extensive and harmonized approach. Every year on average 20,000 deaths occur in Bangladesh due to road traffic accidents while 4,000 are officially reported. These deaths and associated injuries have an immeasurable impact on the families affected and the community as a whole. It has been estimated that Bangladesh loses 1.6% of the GDP every year due to road traffic casualties (WHO, 2013).

It is abundantly clear that national highways are strategically and economically important, since they carry country's most of the traffic, passengers and freight and most importantly have the highest percentage of road traffic casualties in Bangladesh and therefore require urgent attention for improvement. Road infrastructure has been identified as a contributing factor that initializes most of the prevailing categories of road traffic crashes. Numerous researches have also revealed that small investment in infrastructure improvement could prevent substantial number of crashes which might have turned out to be fatal crashes. Assessment of road infrastructure has been proved to be very effective worldwide as it examines the current level of risks and identifies future improvement options.

iRAP assessment process involves a "drive through" inspection capturing video images of the roads and coding the attributes related to road infrastructure and environment. Supporting data like crash rate, AADT, speed etc. in addition with the inspection data result in Star Ratings which is usually represented in colour coded risk maps. Safety investment plan of collective cost-effective countermeasures with benefit-cost ratio indicates the investment required to gain a certain level of improvement of road safety. The detailed methodology of iRAP assessment in Bangladesh has been described in this paper.

The rest of the paper presents the selected results and striking features of 1400 km highway assessment from the iRAP-ADB road safety improvement programme in Bangladesh including star ratings for different road user categories, star rating maps and future safety investment plan. Moreover, it explicates the application of star rating results - how it can be utilized by planners and road safety managers for potential improvements of the highways assessed using iRAP methodology.

2. NATIONAL HIGHWAYS IN BANGLADESH: IMPORTANCE AND RISKS

Roadways are the key mode of transport in Bangladesh and contribute to the country's economy significantly more than other modes like railways, airways and waterways. The major road network of nearly 21,500 km, consisting of national highways along with regional highways and zilla roads, is maintained by Roads and Highways Department (RHD), Bangladesh. National Highways as the most important roadways of the country have connected the national capital to the land port and sea port cities and important district headquarters and thus account for a major part of domestic passengers and freight movements. Despite occupying a small share (16.5%) of the major road network, national highways have the highest percentage of fatalities. Among the police reported 7239 accidents during 2009-2011, 44% have occurred on National Highways. Figure 1 illustrates the share of road network by length and accidents in by roadway type.

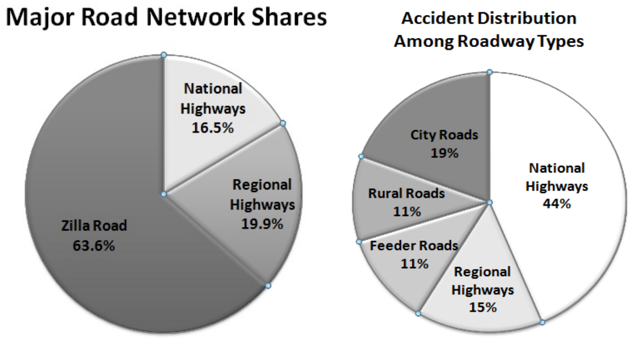

Figure 1: Road Network Share of Roadway Type by %Length (left) and Accident Distribution by Roadway Type (2009-2011) (right)

According to BRTA Annual Report, in 2010, among 1883 fatal accidents occurred, 825 (44%) were on national highways. Distribution of 825 accidents according to types also shows that Hit Pedestrian (47%) is the most predominant accident type followed by Head On (22%) and Rear End (14%). Table 1 represents numbers and percentages of different type of accidents. From the statistics regarding vehicular involvement, it was clear that buses (31%) and trucks (20%) were over-involved in those fatal accidents.

Table 1: Predominant Fatal Crash Types on National Highways in 2010

Crash Type	No. of Crashes	Percentage of Crashes
Hit Pedestrian	390	47%
Head On	181	22%
Rear End	115	14%
Overturn	49	6%
Side Swipe	39	5%
Hit Objects	33	4%
Others	18	2%

The distribution of accidents in Table 1 clearly signifies that small investment in improvement of pedestrian facilities, separation of opposing traffic and treatment of junctions and accesses would reduce the major types of accidents and return huge benefit against their cost. Thus investigation of road infrastructure and road environmental features can be explicitly helpful to have a closer look into the shortcomings and improvement options.

3. THE SAFE INFRASTRUCTURE PERSPECTIVE

Road traffic accient is a multi-factor random event which involves the driver, vehicle and the road including the road environment. Traditionally road safety management put main emphasis on the road users through education and enforcement for improving road safety. For a safe road system, education and enforcement provide an important role to make the road users including drivers more alert, law abiding and competent. But mistakes are likely to happen even if the road users are more careful. It has been widely recognized that self explaining roads with effective road infrastructure could make roads more forgiving and safer. It denotes that safer road infrastructure could minimize not only the risk factors which lead to possibility of crashes to happen and but also the severity of casualties during crashes. When a crash occurs, protective road infrastructure can mean the difference between life and death (Bliss, 2009). Researches have identified infrastructure improvement by creating less vulnerable environment as an efficient way to reduce the number of crashes. For example, Polus finds that a good infrastructure can reduce the crash rates on two way rural highways by 44% (EC, 2004). Therefore, developed countries are focusing on improving road infrastructure as a long term safety option. In Canada's Road Safety Strategy 2015, road environment factors that may affect the likelihood of crash occurrence e. g. roadway configuration, roadway construction, road surface condition, road and roadside design, urban and rural infrastructure have been identified as one of the key issues of collisions being targeted.

4. THE NEED FOR ROAD INFRASTRUCTURE ASSESSMENT PROGRAMME

Traditionally road safety investments in Bangladesh have been targeted on the basis of crash reduction studies like black-spot analysis and treatment. While this approach as a key strategy returned considerable benefits against the costs, but it depends on crashes occurring and people being killed or injured. This urges an approach where crashes can be prevented before it occurs.

Blackspot programmes rely on the use of crash data to identify problematic sites and this logically means that crash data must be comprehensive and precise. Accident database in Bangladesh is poor due to under reporting, lack of proper accident database management system and poor involvement of police to the accident. A study from 2002 estimated that only 49% of the fatal accidents were reported while WHO has estimated it to be several times higher. This can result in problematic sites being overlooked when Roads and Highways Department assemble lists of potential treatment sites and prioritize for treatment. Clearly, when crash data are the only indicators of blackspot treatments, a deficiency in crash data is a serious problem.

Road infrastructure based assessment programme has been proved to be successful in developed countries and now it is time to use this proactive approach in developing countries like Bangladesh. Star Rating is such a proactive methodology, which estimates the risk on the basis of identified deficiencies in road infrastructure rather than crash data, to rate the safety potential of roads. It can be applied to road corridors nationally in a consistent manner, to assess and track road safety performance and can even be implemented in the design. It is a tool that supports the initiatives aimed at providing safer roads and roadsides as outlined in Safe System Approach.

5. INTERNATIONAL ROAD ASSESSMENT PROGRAMME AND BANGLADESH

The International Road Assessment Programme (iRAP) is a registered charity dedicated to preventing deaths by eliminating high risk roads around the world. It has been created to help improve road infrastructure safety in low and middle income countries in order to drive down the global road death toll. iRAP aims towards generating and prioritising large, affordable, high return programmes of safety engineering countermeasures using a globally consistent methodology. iRAP works in partnership with government and non-government organisations to:

- inspect high-risk roads and developing Star Ratings and Safer Roads Investment Plans
- provide training, technology and support that will build and sustain national, regional and local capability
- track road safety performance so that funding agencies can assess the benefits of their investments.

Road Assessment Programmes (RAP) are now active in more than 70 countries throughout Europe; North, Central and South America, Africa and Asia Pacific, working as an umbrella organization for EuroRAP, AusRAP, KiwiRAP, usRAP and ChinaRAP (www.irap.org).

6. METHODOLOGY

iRAP uses an internationally recognised methodology to inspect, assess and plan future improvement option for a high risk road networks.

1. **Risk Maps** represent the level of risk obtained from star ratings for different road users on a road network.
2. **Star Ratings** provide a simple and objective measure of the level of safety provided by a road's design.
3. **Safer Roads Investment Plans** draw on approximately 91 proven road improvement options to generate affordable and economically sound infrastructure options for saving lives.
4. **Performance Tracking** enables the use of Star Ratings and Risk Maps to track road safety performance and establish policy positions.

6.1 What are Star-Ratings?

Star Ratings involve an inspection of road infrastructure attributes that are known to have an impact on the likelihood of a crash and its severity. Between 1 and 5-stars are awarded depending on the level of safety which is 'built-in' to the road. The safest roads (4- and 5-star) have road safety attributes that are appropriate for the prevailing traffic speeds. Road infrastructure attributes on a safe road might include separation of opposing traffic by a wide median or barrier, good line-marking and intersection design, wide lanes and sealed (paved) shoulders, roadsides free of unprotected hazards such as poles, and good provision for bicyclists and pedestrians such as footpaths, bicycle lanes and pedestrian crossings. The least safe roads (1- and 2-star) do not have road safety attributes that are appropriate for the prevailing traffic speeds. These are often single-carriageway roads with frequent curves and intersections, narrow lanes, unsealed shoulders, poor line markings, hidden intersections and unprotected roadside hazards such as trees, poles and steep embankments close to the side of the road. They also do not adequately accommodate for bicyclists and pedestrians with the use of footpaths, bicycle paths and crossings.

6.2 Star-Rating Scores and Risk Mapping

A Star Rating Score (SRS) is calculated for each 100 metre segment of road and each of the four road users, using the equation: SRS = Σ Crash Type Scores
where:

- The SRS represents the relative risk of death and serious injury for an individual road user; and
- Crash Type Scores = Likelihood x Severity x Operating speed x External flow influence x Median traversability

where:

- likelihood refers to road attribute risk factors that account for the chance that a crash will be initiated
- severity refers to road attribute risk factors that account for the severity of a crash
- operating speed refers to factors that account for the degree to which risk changes with speed
- external flow influence (usually AADT) factors account for the degree to which a person's risk of being involved in a crash is a function of another person's use of the road
- median traversability factors account for the potential that an errant vehicle will cross a median (only applies to vehicle occupants and motorcyclists run-off and head-on crashes).
- risk factors have been obtained from years of research in estimating relationships of likelihood and severity of crashes with precise crash record.

A SRS is only produced if a flow of the particular road user is recorded. For example, if no pedestrians are present, then no SRS is produced. SRS are also not produced when major road works are being undertaken. Star Ratings are determined by assigning Star Rating Scores (SRS) for certain ranges for certain road users and are better represented in a risk map.

6.3 Safer Road Investment Plan and Performance Tracking

Safer Roads Investment Plans, which build on Star Ratings to provide a cost-effective, network-wide countermeasure plan for implementation by local stakeholders and funding bodies. Safer Roads Investment Plans involve:

- consideration of the existing condition of the road
- estimates of the number of deaths and serious injuries that occur
- the application of proven engineering countermeasures.

An economic analysis of the proposed countermeasures is undertaken by comparing the cost of implementing the countermeasure with the reduction in crash costs that would result from implementation of the countermeasures. Countermeasures are required to exceed a threshold benefit cost ratio (BCR) if they are to be included in the plan.

iRAP also tracks the changes of star-ratings with the improvement of road network and helps to design a road which maintains a minimum star rating policy target for different road users.

7. RESULTS

With the support from ADB, Roads and Highways Department, Bangladesh (RHD) took initiatives for application of the iRAP inspection methodology for assessing star ratings of national and regional highways.

Details could be seen in iRAP Bangladesh Technical Report, 2013. As a preliminary finding, overall star ratings of the assessed highway of 1372 km are summarized below:

- Vehicle occupants, nearly three quarters (73%) is 1 or 2-star
- Motorcyclists, 81% is 1 or 2-star
- Pedestrians, 97% is 1 or 2-star
- Bicyclists, 92% is 1 or 2-star.

Some of the key features of the detailed conditions of the assessed highways as observed from the survey are:

- 95% of roads are undivided, and many have high overtaking demand
- 20% of the network has hazardous roadsides which is especially critical on curved sections of road.
- Almost three quarters of intersections (70%) are unsignalised with no protected turn lanes and no chanelisation
- 80% of roads where pedestrians are present have no formal footpath
- 96% of roads have no formal bicycle facilities.
- 40% of the network has no paved shoulder.

Detail results of the star ratings, its representation on the maps and star rating scores for each section of the highways are available on the website: vida.irap.org.

8. CONCLUSIONS

Road safety is a major concern in Bangladesh due to the increase in number of accidents in recent times especially on national highways. Road infrastructure has an undeniable role in accidents and systematic road infrastructure improvement level could bring dramatic change in reducing the severity of the hazards and other factors involved that catalyze road traffic accidents. Much still remains to be done in this field to prevent casualties in the future in parallel to the reactive approach like Blackspot treatment. This paper has briefly described the safety situation in national highways and attempted to present internationally recognized Star Rating Road Assessment Programme as an effective road safety management tool to implement in the national level. Some key present infrastructure features along with the star rating results have also discussed. This paper argued that road infrastructure safety assessment as a proactive approach can be considered to be crucial element for achieving the goals and targets of the Road Safety Decade of Action in Bangladesh.

9. ACKNOWLEDGEMENTS

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REFERENCES

Bangladesh Road Transport Authority (BRTA) (2010), Statistics on Registered Motorized Vehicles in Bangladesh.

Bliss, T. (2009), “Safe Roads for Development: A Policy Framework for Safe Infrastructure on Major Road Transport Networks”, World Bank, May 2010

Canada's Road Safety Strategy 2015 (2011), Canadian Council of Motor Transport Administrators

European Commission (2004), Road Safety Impact Assessment Report

Hoque, M. M., Smith, G., Rahman M. A., Uddin, M. H. M. (2012), “iRAP and Road Infrastructure Safety Assessment in Bangladesh”, Australasian Road Safety Research, Policing and Education Conference 2012

Hoque, M.M., Rahman M. A. and Smith G. (2013), “Quantifying Highway Safety Hazards in Bangladesh", Road Safety Education, Research, Policing and Education Conference, 2013.

International Road Assessment Program (2013), Bangladesh Technical Report.

www.irap.org, Website of International Road Assessment Programme (iRAP).

BRT AS MASS TRANSIT OPTION: THE CONTEXT OF DHAKA

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ABSTRACT

Dhaka the Capital City of Bangladesh suffers from critical and deteriorated traffic congestion largely due to the absolute lack of roads, deficient road network configuration and inefficient traffic management (JBIC, 2000). Existing public transport system, bus transit operations in particular is characterised as far short of the desirable mobility needs of the people in terms of reliability, comfort speed and safety. Buses are generally considered unreliable and time consuming to reach the destination. Thus, there is need to develop a system to give priority and dedicated road space to buses in order to make them reliable and faster. Developing countries with high transit-dependent populations and limited financial resources have increasingly attempted the use of BRT systems because of their low costs and relatively fast implementation times. The cost of a BRT project is considered to be approximately one-third of a LRT project, which is a cost that developing countries can afford.This paper introduces the concept of BRT system and discusses its relevance and potential to enhance and improve the quality of public transport services in metro Dhaka. The paper has been based on the extensive review of the published literature and the information gleaned by the authors from the ongoing BRT Project in Dhaka.

Keywords: *Mobility, Public Transport, Traffic Congestion, Bus Rapid Transit, Transit Solution.*

1. INTRODUCTION

In spite of low level of motorisation, Dhaka the Metropolitan City of Bangladesh suffers from critical and deteriorated traffic congestion largely due to the absolute lack of roads, deficient road network configuration and inefficient traffic management (JBIC, 2000). Existing public transport system, bus transit operations in particular is characterised as far short of the desirable mobility needs of the people in terms of reliability, comfort speed and safety. In Dhaka, buses are generally considered unreliable and time consuming to reach the destination. Thus, there is need to develop a system to give priority and dedicated road space to buses in order to make them reliable and faster. Various projects around the world have indicated that Bus rapid transit (BRT) is an effective alternative for congested cities at a relatively low construction and operation cost and is considered by the policymakers and public transport specialists as an effective way to improve bus transit services. It is urged that Bus Rapid Transit (BRT) has been seen as a "creative, emerging public transit solution" which can be cost-effective in addressing urban congestion *(Currie, 2006, Levinson et al. 2003, U.S. General Accounting Office 2001*).

Today, the BRT concept has increasingly implemented by cities looking for cost-effective transit solutions and has emerged as an economical transit alternative with significant potential for developing countries. The purpose of this paper is to introduce the key transport and traffic characteristics in metro Dhaka and to discuss the potential of introducing BRT as a mass transit option in metro Dhaka to cater for ever increasing public transport demand towards alleviation of congestion level and achieving a sustainable public transport system. The paper argues that the introduction of BRT will have significant role to enhance and improve the quality of public transport services in metro Dhaka.

2. OVERVIEW OF TRANSPORT SYSTEM IN DHAKA

2.1 The Context

Dhaka, the capital city of Bangladesh with current population of 17 million has been growing at astonishing levels since the independence. Its metropolitan area is home to almost 15 million people in an area of 1,528 km² (about 17 million in the Greater Dhaka). By 2020, the megacity's population is expected to rise to 20 million people. It is also one of the most densely populated cities in the world, with more than 45,000 people per square meter in the core *area (Consultant's Report, Preparing the Greater Dhaka Sustainable Urban Transport Corridor Project, 2011)*. Per capita income averages around US$ 900 per year, and around 30 percent of the population lives in miserable conditions, with very poor access to transport services *(Bangladesh Economic Review, 2011).*

2.2 The Problem Characteristics

The rapid urbanisation process, high vehicular population growth and that of the mobility, inadequate transportation facilities and policies, varied traffic mix with over concentration of non-motorised vehicles, absence of dependable public transport system and inadequate traffic management practices have created a significant worsening of traffic and environmental problems in the metropolitan Dhaka. Road traffic congestion continues to remain a major problem and indeed is deteriorating rapidly resulting in massive socio-economic losses. The greater challenge thus for transportation professionals is to develop a system of urban transport that meet the basic mobility needs for all urban dwellers at desirable safety and avoiding the unacceptable level of congestion and its consequent overwhelming adverse environmental effects.

The transport system in Dhaka includes many different modes of travel - both motorized and non-motorized. These diverse modes often use the same road space, resulting in a high level of operational disorder. The city's transport environment and system are unique among cities of comparable size in the world, being predominantly road based with a substantial share of non-motorized transport. Buses and minibuses, the cheapest mode available as mass transit, are constrained by poor service conditions: long waiting, delay on plying, overloading and long walking distance from the residence/work place to bus stoppages are some of the problems that users confront daily. This situation has resulted in deterioration in accessibility, level of service, safety, comfort and operational efficiency, causing increased costs, loss of time, air pollution and psychological strain, and posing a serious risk to the economic viability of the city and the sustainability of its environment.

In addition, the city's road space is limited, with few alternative connector roads, lacking of effective maintenance and management, most of it with geometrical conditions that make them not accessible to buses. With non-motorized transport as a significant mode, there are no effective bi-cycle lanes and safe walkways, and the footpath available for pedestrian is occupied in great proportion by vendors and others. Most of signals are manually controlled and police have to control traffic, without properly coordinated automated systems. With policy formation and control shared between governments agencies poorly coordinated, there has been a lack of organized effort to handle the situation (ALG 2012). There is now an ever-increasing urgency for mitigating the complex transportation problems in Dhaka by the augmentation of mass transit modes (Hoque and Hossain, 2004).

2.3 Vehicular Growth

In Bangladesh, motorised traffic is growing rapidly, around 300 new motorised vehicles are coming to road every day. The number of registered motorised vehicle grew from 7,37,400 in 2003 to 17,51,834 in June, 2012. More than 40% of all registered vehicles (7,08,197) are in Dhaka (BRTA, 2012). Trends of motor vehicle growth are shown in Figure 1.

It can be observed from the Figure 1 that, the number of privately owned motor vehicles particularly motorcycles and cars are growing rapidly which increased by 200% and 250% respectively over the period of 8 years. Motorcycles constitute around 42% of total motorised vehicles. Public transport such as buses and minibuses has not grown substantially despite the demand for public transport services has increased considerably. There are 11,060 buses and 8,583 minibuses plying on roads which represents only about 3% (buses and minibuses combined) of total motorised traffic. The share of bus fleet (buses and minibuses combined) has been in fact declining (see figure 2). Cars and motorcycles are becoming increasingly necessity to get around, especially given the unsatisfactory alternative of slow, overcrowded, undependable, and dangerous public transport services (Pucher et al., 2005).

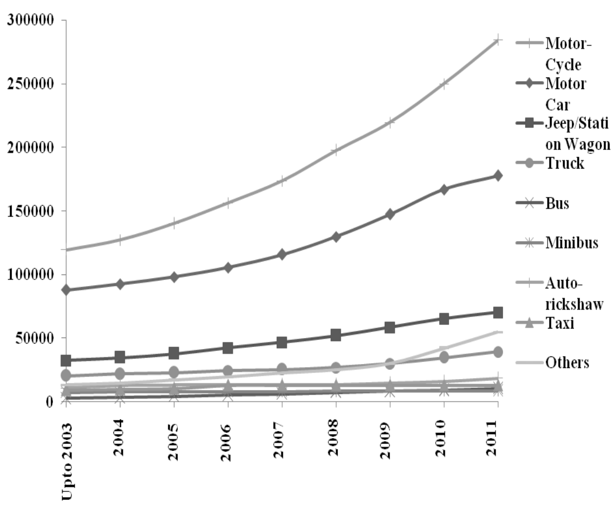

Figure 1: Motorized vehicle growing trend in Dhaka (Hoque et al, 2012)

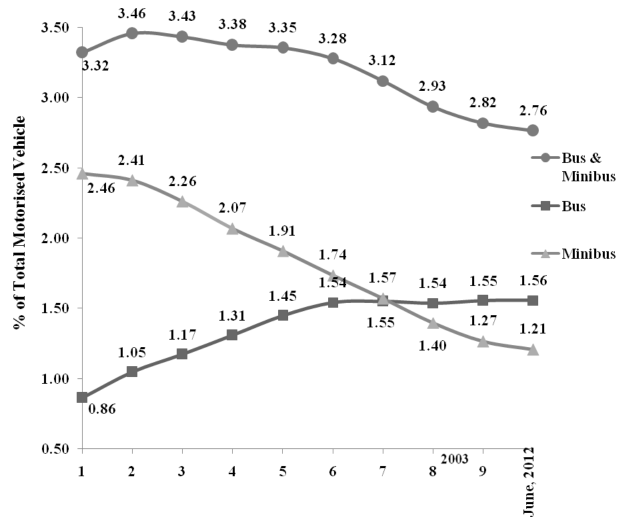

Figure 2: % Growth of buses and minibuses in Dhaka (Hoque et al, 2012)

The latest BRT study *(Advance Logistics Group Study, 2011)* estimated that on an average day 21 million trips are taking place in Dhaka metropolitan area. Despite the rapid growth of motorised traffic in Dhaka, non-motorised transport still remains the dominant mode for the city dwellers who are mostly middle and low income groups. More than 40% of the city trips (see Table 1) are served by walking and rickshaw *(Final Report, Dhaka Urban Transport Network development Study, DHUTS, 2010*). The varied traffic mix and heavy concentration of non-motorised vehicles with almost 70% of the available road space is occupied by rickshaws and their dominance is expected continue in the foreseeable future *(Hoque, M. M., 2004)*. Currently, rickshaw movements are however restricted in some major roads.

Table 1: Modal share in metro Dhaka

Mode	Percentage of Share				
	DITS *(1994)*	DUTP *(1997)*	JBIC Study *(1999)*	STP *(2005)*	JICA Study *(2009)*
Walk	60.1	62.82	62.05	14.0	19.09
Rickshaw	20.1	20.04	13.28	34.0	38.19
Bus	12.8*	10.42*	10.22	44.0*	29.83
Auto-rickshaw			5.83		5.73
Passenger Car	7.0**	6.72**	3.97	8.0**	4.30
Others			4.65		2.86
Total	100	100	100	100	100

* Transit
** Motorized (Non Transit)

Some striking features found from the survey conducted for '*Dhaka Urban Transport Network development Study (DHUTS),* (2010) project can be acknowledged as:

- Low income (monthly salary <12500Tk) people make 73% trips on foot.
- While most of the rickshaw trips are made by the middle income (monthly salary 12500-55000Tk) People (59%).
- Walk and rickshaw trips relate to 97% of the city dwellers.

Around 30% of the total travel are attributed to bus trips *(ALG Study, 2011)*. Buses comprise only 9.7% of the vehicle mix that combines all vehicles and pedestrians, but bus passengers account for 77% of all public transport users *(Final Report, Study on Bus Operation in Dhaka City, 2007).* However, Dhaka, is perhaps the only city of its size without a well-organized, properly scheduled bus system or any type of mass rapid transit system. Much is needed to be done to serve existing transport needs better. The challenge is to establish an overall framework for a multi-modal transport system that effectively serves current and future land uses and satisfies demand to the greatest extent possible.

3. BUS TRANSIT IN METRO DHAKA: MAJOR ISSUES OF CONCERN

There are many concerns related to the current bus service as per the information and the feedback obtained by different studies and surveys, among them are:

- <u>Traffic Operational hazards:</u> High level of operational disorder, rapid population growth and the absence of proper planning and control considerably diminishes the efficiency and effectiveness of the existing transport systems. Some striking features are:
  - Buses stop anywhere to take passengers at will without using recognized bus stops.
  - Many drivers do not abide by basic driving rules and regulations.
  - Ineffective enforcement of traffic laws and widespread disregard of regulations.
  - Poor traffic engineering practices such as road markings, signs, traffic signals, turn restrictions etc.
  - Haphazard crossing of roads by pedestrians not only at intersections but also at mid-blocks of the roads.
  - Pedestrian walkways are either non-existent or in poor physical condition, or blocked by various obstacles, thus forcing pedestrians to walk on the road.
  - Poor road surface conditions and inadequate drainage system that cause disruption to the smooth flow of traffic as well as are safety hazards.
  - Parking on the roads thereby reducing capacity.
  - Encroachment by vendors and hawkers who illegally occupy public spaces causing negative impacts to traffic operations on roads.

- <u>Safety Concern:</u> The vulnerable road users, pedestrians in particular are at serious risk. Some of the major safety issues are:
  - *Irresponsible and careless driving* – excessive speed; overtaking without proper precautions; overloaded vehicles, talking over the cell phones and allowing passengers on the roof-top.
  - *Poor road geometry and poor condition of vehicles* – insufficient road width; sharp bends; and narrow bridges; brake failures; and lack of proper maintenance.
  - *Poorly trained drivers* - a large number have only fake or no licenses, and are often poorly trained, unfamiliar with basic traffic laws and often act improperly.
  - *Careless movement of pedestrians* – at traffic intersections in urban areas; and around market places.

- <u>Service and Behavioural Issues:</u>
  - Neither the bus service providers nor the bus service users follow any norms to improve the current pitiable situation.
  - Lack of communication in the existing bus services. The ticket counters cannot inform the passengers exactly when the next bus is arriving at the particular counter and they are not equipped to communicate with the buses.
  - For most of the buses and minibuses, fare collection is conducted generally inside the bus which very often creates chaotic situation. On the other hand, at some bus stops, there are too many ticket booths, depending on the number of major operators serving on that route.
  - Traffic police sometimes stop buses and negotiate different issues until they are satisfied. This leads to enormous sufferings of the passengers.
  - Little care about the quality of bus services by the operators, companies, drivers and conductors.

4. BUS RAPID TRANSIT (BRT) IN THE CONTEXT OF DHAKA

Revitalization of public transport is a core issue in the context of rapid motorization. Improving quality of public transport, increasing public transport capacity and thus relieving traffic congestion are the significant strategies. There are several options in mass transit facility including Bus way; Tram, LRT (Light Rail Transit) and Metros *(Hoque M. M., 2004)*. With relative advantages, BRT option is seen as urgent consideration for Dhaka.

Indeed, Bus Rapid Transit (BRT) combines the benefits of light rail transit with the flexibility and efficiency of bus transit. Cities in developing countries have struggled with the problem of how to upgrade and improve existing transit services at a low cost. Developing countries with high transit-dependent populations and limited financial resources have increasingly attempted the use of BRT systems because of their low costs and relatively fast implementation times. The cost of a BRT project is considered to be approximately one-third of a LRT project, which is a cost that developing countries can afford. After construction the system is practically self-financing with fares of about $US0.50 per trip. BRT has proved that it allows low fares and reduced travel times for low income users (Leal, M., Bertini, R.L., 2003).

Figure 3: Examples of successful BRT projects

Some of the reasons for choosing BRT over the other options can be cited as:

- Given the costs and community impacts associated with major road construction, improved and expanded public transit emerges as an important way to provide the needed transportation capacity.
- BRT can often be implemented quickly and incrementally without precluding future rail investment if and when it is warranted.
- For a given distance of dedicated running way, BRT is generally less costly to build and equip than rail transit. Moreover, there are relatively low facility costs where BRT vehicles operate on existing bus-only or in mixed traffic lane.
- BRT can be cost-effective in serving a broad variety of urban and suburban environments. BRT vehicles, whether driver-steered or guided mechanically or electronically, can operate on streets, freeway medians, railroad rights of-way, arterial structures, and underground. BRT can easily and inexpensively provide a broad array of express, limited-stop, and local all-stop services on a single facility, unlike most rail systems.
- BRT can have relatively low operations and maintenance costs. This is primarily because the relatively low fixed maintenance costs can offset variable driver costs.
- BRT is well suited to cost-effectively extend the reach of existing rail transit lines by providing feeder services to areas where densities are currently too low to support rail transit. It can also serve as the first stage for an eventual rail transit line.
- BRT can be integrated into urban and suburban environments in ways that foster economic development and transit- and pedestrian friendly design. Examples of regions that have integrated BRT successfully include Curitiba, Bogota, Jakarta, Guangzhou, etc (Hoque et al, 2013).

The *Strategic Transport Plan (STP, 2005), Dhaka Transport Coordination Board (DTCB),* was focused predominantly on formulating strategies for the development of transport infrastructure over the next 20 years, also including a road management program and recommendations for the establishment of a new Unitary Authority to integrate transport and land-use planning at a strategic level. STP underscores the large size of the transport investment needs in Dhaka and recommends a program that includes three Bus Rapid Transit (BRT) routes, three metro rail systems and fifty highway projects, including construction of a 29 km elevated expressway system, with a total investment of US$ 5.5 billion.

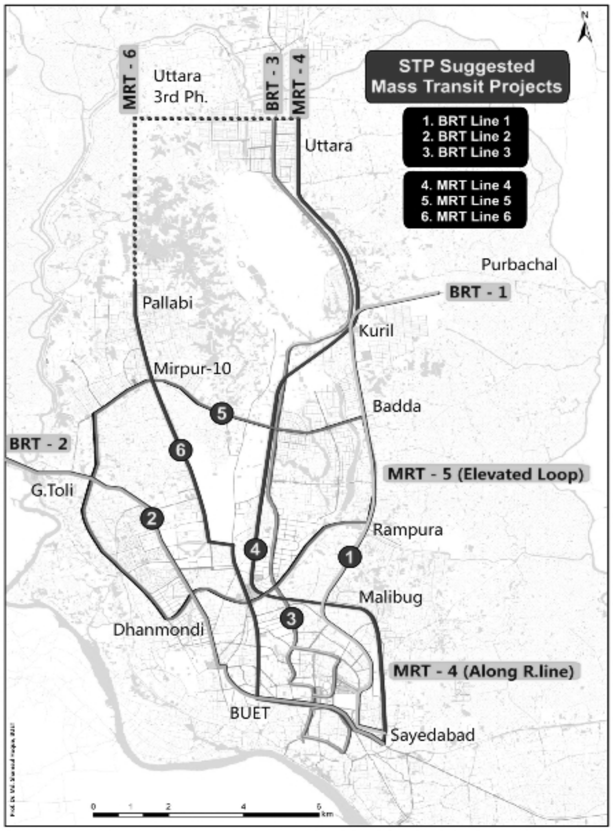

Figure 4: BRT and MRT Corridors proposed

It has suggested the development of six major corridors as mass transit routes as a means for achieving sustainable urban transport in the city. Three radial corridors as follows are thought to be potentially suitable for Bus Rapid Transit (BRT) introduction.

Corridor A : Starting in Uttara in the north and following Dhaka Mymensing Road, Pragati Road, DIT Road Toyenbee Circular Road to Saidabad Bus Terminal.

Corridor B : Starting at Gabtalli and following Mirpur Road, Zahir Raihan Sharani Road to Saidabad Bus Terminal

Corridor C : Starting at International Airport following Airport Road, Shaheed Tazuddin Road and ending in Ramna area.

These three BRT corridors are now actively been considered by the government with the support from the development partners such as the World Bank and the Asian Development Bank. The authors have been playing important role in developing the preliminary design phase of the BRT Line-3 (Corridor C). Some contemporary issues are briefly outlined.

5. SOME POSITIVE CONSEQUENCES OF BRT IN DHAKA

- *Improved Travel Performance:* Decrease bus and general purpose traffic journey times in the corridor. Improvements in connectivity from the corridor to all parts of the city, through high-quality interfaces with connecting buses and rickshaws.

- *Improvements in attractiveness of public transport:* Promotion of service that attracts new users with alternative private transport options and that offers enhanced services to those who depend on public transport.
- *Congestion reduction:* Shifts in market to public transport. Improvements in traffic behavior through system design, improved management and effective enforcement techniques that lead to reductions in congestion.
- *Supporting special needs groups:* Overall increases in mobility and specific improvements in availability and quality of mobility for the urban poor. Improved trip and comfort characteristics for special needs groups such as the elderly, handicapped and women in general.
- *Industry support:* Provide opportunities for effective private sector participation support for existing bus and rickshaw industries.
- *Keeping costs reasonable:* A relatively low-cost of implementation sensitive to the local needs of Dhaka.
- *Effective regulation:* Develop an effective and accountable system of regulation to select BRT operators; set fares; establish equipment, performance, and good governance standards to be met be franchise holders.
- *Enhancement of management:* Build capacity to manage corridor transport systems. To develop an improved public transport management regime through new institutional and organizational frameworks to manage the corridor transport systems and to develop a high level of service efficiency in the corridor through optimized scheduling and bus speeds that maintain a high level of fleet and passenger efficiency.
- *System integration:* Better synchronisation of the public transport systems with road and infrastructure development.
- *Safety enhancement:* Overall increase in safety in the corridor through the design of system elements and enforcement approaches that promote safety.
- *Pollution reduction:* Reducing environmental pollution and ensuring World Bank's social and environmental safeguard policies not only in construction period but also when the BRT is in full operation.
- *Enhancement of facilities for pedestrians:* Augmentation of pedestrian sidewalks and overbridges/crossings throughout the corridor, for public access in general.
- *Urban environmental management/ Landscape Planning:* Integrate transport more effectively with land uses and to improve the urban environment. The project will take a proactive environmental stance in developing a strong environmental improvement mandate.

6. CONCLUSIONS

Review of published literature revealed that Bus rapid transit (BRT) is considered by the policymakers and public transport specialists as an effective way to improve bus transit services. Developing countries with high transit-dependent populations and limited financial resources have increasingly attempted the use of BRT systems because of their low costs and relatively fast implementation times. BRT can be the solution of these problems as it is a cost-effective mode of transportation and is gaining its increasing popularity worldwide. It has emerged as an economical transit alternative for developing countries and its introduction has recently accelerated in Asia. Poor traffic management, lack of road spaces and the absence of organized public transport resulted in severe traffic congestion, massive delays, increased fuel wastage and resource losses are critical features of the transportation problem in Dhaka. There is thus an ever increasing urgency of introducing BRT system in mitigating the complex transport problems in Metro Dhaka. This paper has introduced the concept of BRT system and discusses its relevance and potential to enhance and improve the quality of public transport services in metro Dhaka.

7. ACKNOWLEDGEMENTS

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8. REFERENCES

ADB (2011), "Preparing the Greater Dhaka Sustainable Urban Transport Corridor Project", Consultant's Report.

Bangladesh Road Transport Authority (2012), "Number of Yearwise Registered Motor Vehicles in Dhaka", Available at: http://www.brta.gov.bd/images/files/motor_v _dhaka_05-08-12.pdf.

Bangladesh Economic Review (2011), Economic Adviser's Wing, Finance Division, Ministry of Finance.

Bhuiyan, A. A. (2007), Final Report, "Study on Bus Operation in Dhaka City", "Air Quality Management Project (AQMP)", Department of Environment (DoE), Ministry of Environment and Forest (MoEF).

Currie, G. (2006), "Bus Rapid Transit in Australasia: Performance, Lessons Learned and Futures", Journal of Public Transportation, Volume. 9, No. 3, National Center for Transit Research, University of South Florida.

Dhaka Transport Co-ordination Authority (DTCA) and Advanced Logistics Group (ALG) (2011), "BRT and Corridor Restructuring Implementation Study and Preliminary Design Work for the Uttara–Mohakhali–Ramna Sadar Ghat Corridor in Dhaka", Contract for Consultants Services between DTCA and ALG.

Dhaka Transport Co-Ordination Authority (DTCB) (2010), 'Dhaka Urban Transport Development Study (DHUTS)', Final Report, Bangladesh University of Engineering and Technology (BUET) and Japan International Cooperation Agency (JICA) Study Team.

Hoque, M. M. and Hossain, T (2004), "Augmentation of Mass Transit Mode in Dhaka, Bangladesh", Proceedings of CODATU XI: World Congress: Towards More Attractive Urban Transportation, Bucharest , 2004.

Hoque, M. M., Barua, S., Ahsan, H. M., and Alam, D. (2012), ‘BRT in Metro Dhaka: Towards Achieving a Sustainable Urban Public Transport System’, Proceedings of CODATU XV: The Role of Urban Mobility in (re)shaping Cities, Addis Ababa, Ethiopia.

Hoque, M. M., Alam, D., Mamun, S. M. A. A., and Ahsan, H. M. (2013), ‘Bus Rapid Transit (BRT): What is it and Why Do We Need it for Dhaka’, Proceedings of 3rd International Conference on Urban Public Transportation Systems, ASCE, Paris, France.

Japan Bank for International Co-Operation (JBIC) (2000), 'Study of the Improvement of Transportation and Environment in Dhaka', Dhaka, Bangladesh.

Leal, M., Bertini, R.L. (2003), "Bus Rapid Transit: An Alternative for Developing Countries", Compendium of Technical Papers, Institute of Transportation Engineers Annual Meeting 2003, Seattle, Washington.

Pucher, J, Korattyswaropam, N., Mittal, N. and Ittyerah, N. (2005), "Urban transport crisis in India", Journal of the 'World Conference on Transport Research Society', Volume. 12, Issue. 3.

STP (2005), “Strategic Transport Plan for Dhaka”, Draft Report, Prepared by Louis Berger Group and Bangladesh Consultant Ltd.

CPEDESTRIANS IN SUSTAINABLE URBAN TRANSPORT MANAGEMENT AND CLIMATE MITIGATION: THE CASE OF MEGACITY DHAKA

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ABSTRACT

Pedestrian is a mode of transport that contributes to reduce motorized and non motorized traffic demand as well as minimize carbon emission and induce growth of green urban infrastructure. It mitigates the impacts of changed climatic situation. As Dhaka is a rapidly growing spontaneously developed city, it lacks proper transportation planning. About 66% of all work trips in Dhaka are on foot. But unfortunately pedestrian infrastructure and facilities are poorly devised. Pedestrian crossing phase at signalized intersection has not yet been introduced. In Dhaka City Corporation (DCC) Area there exists only 320.44 kilometers of footpaths having average width of 1.96 meter. Most of them are not partly used by the pedestrians and are kept open by the street vendors as per their willingness. As pedestrian has not been socially recognized as a mode of transportation, it also gets less priority in providing crossing facilities. The study reveals that 70% intersections have no visible cross marking, 73% of them have no foot over bridge or underpass, 30% are not provided with refuge islands. This study focuses on the need of pedestrians in sustainable urban transport management induced to climate mitigation for Dhaka city in the context of Bangladesh.

Keywords: *Urban transport management, exclusive pedestrian phase, green street, climate mitigation*

1. INTRODUCTION

Pedestrians may be defined as those human traffic who are supposed to walk as a part of their movement and to use those facilities such as footpath, overpass, zebra crossing, subway etc. at any stage of their travel in order to accomplish their activities with which they are engaged (Jannat, 2011). Pedestrian activity is a major component in urban street capacity analysis and pedestrian characteristics are an important factor in the design and operation of transportation systems. Roads sector makes 15% of the total greenhouse gas emissions (Hossain et al., 2012). In this context encouraging pedestrian can not only contribute to reduce travel demand of using cars but also physical movement and good exercise as well as contributing climate change mitigation.

Cities are home to over half of the world's population and are at the forefront of the climate change issue. The increase in motorized traffic has resulted in an increase of air pollution, traffic congestion and a reduction in the quality of life for urban dwellers. The activities of motorized transport industry release several million tons of harmful gases each year into the atmosphere such as lead (Pb), carbon monoxide (CO), methane (CH4), nitrogen oxides (NOx), sulphur dioxide (SO2) etc. and particulate matters (ash, dust). Some of these gases participate in depleting the stratospheric ozone (O3) layer which naturally screens the earth's surface from ultraviolet radiation which has a very severe impact on climate change (Munni et al., 2013).

Urbanization in the modern sense characterizes the shift that happens when a society moves from primarily being an agrarian society to becoming an industrial and knowledge based (Ha, 2013). By 2050, more than half of population in developing countries will live in cities. This is the transition time for Bangladesh for capacity building of future policies how to merge with the globalization process and how the future cities should be formed and structured. Pedestrianization policy is to be incorporated in planning and development of future road networks by all the concerned stakeholder departments. It may be Local Government Engineering Department (LGED) in constructing rural road encircling a growth centre, may be Roads and Highways Department (RHD) when the national highway intersects a small town, may be RAJUK, City Corporations, Pourashavas or National Housing Authority (NHA) in designing residential land projects for greening future cities.

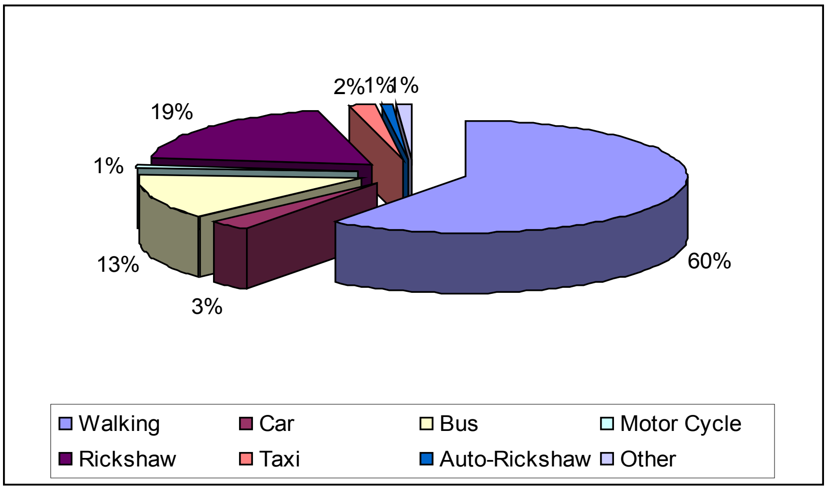
Most of the major transportation corridors in Dhaka are north south oriented. The proposed Mass Rapid Transit (MRT) routes are also designed along north-south road. East-west elongated roads mainly act as connectors. The study on pedestrians in sustainable urban transport management and climate mitigation reveals analysis on the emergence of pedestrian facilities which will be helpful for future policy formulation regarding efficiency and safety improvement of the pedestrian infrastructure.

2. PEDESTRIAN AS ONE OF THE DOMINANT MODE OF TRANSPORT IN DHAKA CITY

There is a wide variety of transport modes available in Dhaka city. Walking is a commonly used mode of transport. The proportion of trips made by walking is substantial and for some people, walking is a matter of choice and convenience. However, the reality is that for many people, walking is a matter of economic necessity (RAJUK, 1995). Beside the pedestrians, other means of travel are bicycle, rickshaw, motorcycle, baby-taxi, tempo, minibus, bus, car, taxi, jeep etc.

Despite a high dominance of 'pedestrian' as a mode of transport, facilities have been provided, only been added as an afterthought to road improvements. It has been estimated that there are only about 400 kilometres of footpath within the Dhaka City Corporation (DCC) area. Where footpaths have been built, there are frequent obstructions that block or otherwise reduce their overall usefulness. Such obstructions include: -

- Temporary vender stations and hawkers who occupy portions of the footpaths;
- Parked cars;
- Solid waste skips;
- Building materials and debris that are stored or abandoned on the footpath; and
- Holes, surface irregularities and water accumulation.



Source: RAJUK, 1995.

Figure 1: Percentage of different Modal Vehicles of Dhaka City in 2003

In Dhaka city, nearly 40% percent of the footpaths are being occupied illegally (DTCB, 2007). In spite of a High Court ruling on February 11, 2001 ordering that the responsible agencies make all footpaths free from illegal occupation, no significant change or improvement is evident. As a consequence, pedestrians are often forced to walk in the street instead of on the footpaths, even in areas where footpaths are provided. Pedestrians walking on the road increase the risk of traffic-related pedestrian injuries and also have the adverse effect of the reducing the capacity of the road and thereby increasing congestion. Available information indicates that pedestrians are involved in half of all road collisions in the city. Two-thirds of all traffic related fatalities are pedestrians. According to STP survey, pedestrian volumes of 10,000 to 20,000 per day are common and reach as high as 30,000 to 50,000 per day. During the peak hour pedestrian counts of 1,000 to 3,000 per hour are common and reach as high as 5,000 in some areas (Jannat, 2011).

2.1 EXISTING CONDITION OF FOOTPATH

There are only about 400 kilometers of footpaths within the DCC area, compared to a road network of 1,293 km (Devcon, 2009). Ideally, footpaths should exist on both sides of a street; this suggests that Dhaka should have almost 2600 km of footpaths. Only 37% of observed roads had footpaths on both sides, and almost half had none at all. Only 15% of all footpaths were free of obstructions. In 65% of the observed segments, the observer had to leave the footpath at least once because of obstructions (Munni et al., 2013). Footpaths are remained dirty, occupied be vendors and sometimes wastes are kept partly covering footpaths creating ugly public nuisance. Urination on the footpath, waste from vendors, waste water flow, dust and remains from construction materials etc. make the footpath dirty. It psychologically demoralizes pedestrians to use the footpaths.

Plate 1: Waste dumping on the footpath

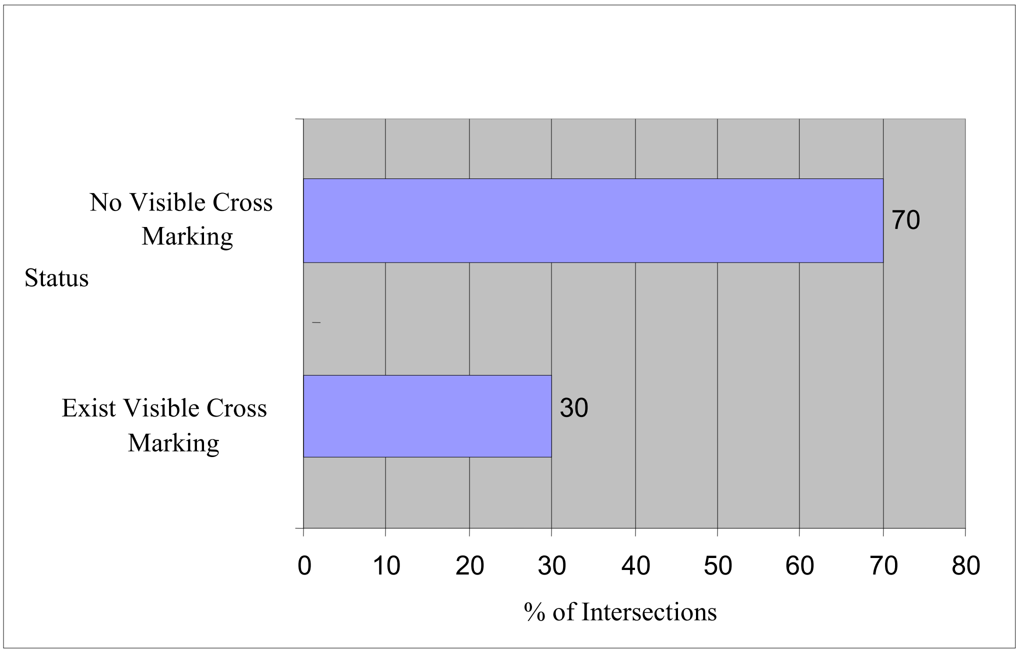
2.2 PEDESTRIAN CROSSING BEHAVIOUR

All road users are pedestrians at some stage of each journey and some are pedestrians the whole time (Waresh, 2001). Pedestrian crossing behavior depends on individual socio-demographic factors such as gender and age; environmental factors such as land use, street design, and facilities such as pedestrian overpasses and underpasses; traffic conditions such as heavy, high-speed traffic; enforcement factors; and factors related to education and policy (Mehndiratta et al., 2010). The movement of people and goods are linked with distribution and intensity of land use. People move because of activities with which they are engaged. As such pedestrian is, therefore function of activities. Pedestrian issue is important especially in urban areas where intense land use cause concentrated activities. They are the most difficult transport group to control and hard to enforce. The pedestrians are more often disobey the traffic control devices than the drivers (Jannat, 2011).

3. EXISTING PEDESTRIAN FACILITIES

3.1 Visible Cross Marking

A pedestrian crossing or crosswalk is a designated point on a road at which some means are employed to assist pedestrians wishing to cross. They are designed to keep pedestrians together where they can be seen by motorists, and where they can cross most safely across the flow of vehicular traffic. Pedestrian crossings are found at intersections, and also at other points on busy roads. They are generally installed in the points where large number of pedestrians are attempting to cross (such as in shopping areas) or where vulnerable road users (such as school children) regularly cross etc. In Dhaka city area, about 70% intersections have no visible cross marking (Jannat, 2011).

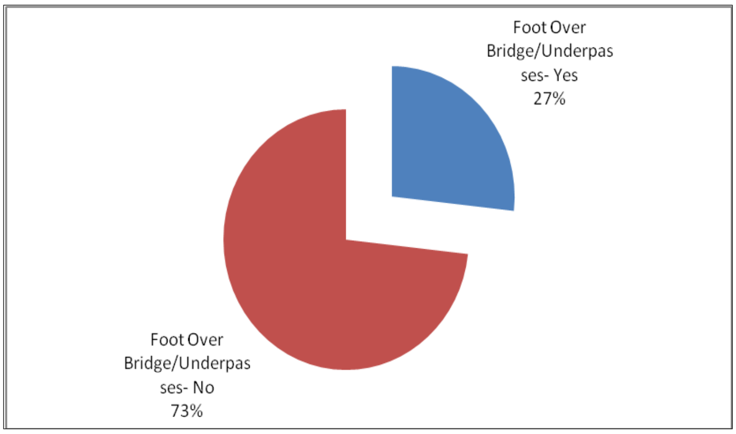


Source: Jannat, 2011.

Figure 2: Percentage of intersections having visible cross mark

3.2 Foot Over Bridge and Underpass

Pedestrian overpass and underpass are grade separated crossing. These allow the pedestrians safe crossing over busy roads without impacting traffic. In most cases pedestrians choose at grade crossing. In Dhaka city area about 27% intersections have foot over bridge and underpass whereas about 73% intersections have no foot over bridge and underpass (Jannat, 2011).

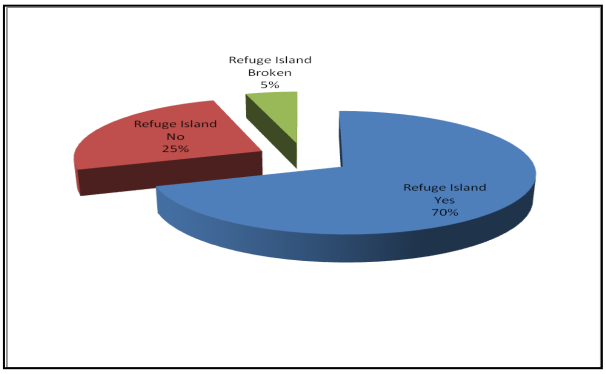


Source: Jannat, 2011.

Figure 3: Intersections having foot over bridge and underpasses

3.3 Refuge Island

A refuge island, also known as a pedestrian refuge or pedestrian island, is a small section of pavement or sidewalk, where pedestrians can stop before finishing crossing a road. It is typically used when a street is very wide, as the pedestrian crossing can be too long for some individuals to cross in one traffic light cycle. In Dhaka city the study reveals that about 70% intersections have refuge islands, about 5% are broken and about 25% intersections have no refuge island (Jannat, 2011).



Source: Jannat.

Figure 4: Percentage of intersections having Refuge Island

4. PROBLEMS ASSOCIATED WITH PEDESTRIAN CROSSING AT INTERSECTIONS

One of the major problems associated with pedestrian crossing is lack of visible cross marking among the existing intersections of Dhaka city. Only 30% of the signalized intersections are provided with visible cross marking. This is a great threat to the safety of pedestrian crossing in Dhaka city.

There exist inadequate footpaths and crossing facilities in most of the areas. In many cases, the footpaths are in poor physical condition with inadequate crossing facilities. There is possibility with multiple threat collision because there is not enough stop bar/ speed breaker to stop back far enough from the crossing points. It was found that a quarter of the surveyed intersections are not provided with any median and refuge island where people cross the road in unsafe way (Jannat, 2011). Median island is necessary to split the traffic in two groups to cross the road. In many cases illegal parking is found near the visible cross marks which obstacles pedestrian crossing and is a threat for safety. Street lighting around the crossing is found to be inadequate. As a result the visible cross marks are not easily captured by the vehicle drivers to have stopping sight distance to avoid collision.

5. INTRODUCING EXCLUSIVE PEDESTRIAN PHASE (EPP) IN DHAKA

An Exclusive Pedestrian Phase which is known as a pedestrian scramble is a pedestrian crossing system that stops all vehicular traffic and allows pedestrians to cross an intersection in every direction, including diagonally, at the same time.

Plate 2: Pedestrian scrambles at Hachik Square, Shibuya, Tokyo

It is also known as a 'X' Crossing in UK, diagonal crossing in USA, scramble intersection in Canada, and more poetically it is also called as Barnes Dance. It was first used in Canada and in the United States in late 1940's and has since then been adopted in many other cities and countries. The understanding of the benefits in terms of pedestrian amenity and safety has led to new examples being installed in many countries in recent years. The most famous implementation of this kind of intersection is present in Shibuya, Tokyo which has similar urban formation characteristics as in Dhaka.

5.1 EPP in New Market Intersection

A previous study conducted by Hasinae-Jannat designed a model of pedestrian phasing in some intersections of Dhaka city. One of them namely pedestrian phasing in New Market intersection has been provided below which is interesting and can be introduced on pilot basis.

Saturation Flow rate=3575
Lost time Per Phase =2.4 sec
Amber time is =3 sec
Assume v/c ratio =0.9
Saturation Headway, h =3600/s =3600/3575 =1.01

Sum of critical lane volume is the sum of maximum lane volumes in each phase

Vci =1162+674+128+1007 = 2971

Cycle length can be found out from the equation $C = \frac{N*L*(v/c)}{(v/c - vc/si)}$

Where

N = no. of phase
L = Lost time per Phase
v/c = Critical volume capacity ratio

$$C = \frac{4*2.4*0.9}{(0.9- 2971/3575)}$$

C = 125.3 sec

The effective green time can be found out as

Gi =Vci/Vc (C-L)

The effective green time =(125.3-2.4*4)
=115.7 sec

Green splitting for different phase can be found out as

G1= 115.7* 1162/2971 =45.25 sec
G2= 115.7* 1007/2971 =39.31 sec
G3= 115.7* 128/2971 =5 sec
G4= 115.7* 674/2971 =26.24 sec

Similarly actual green time for phase 1, G1=45.25 -3+2.4= 44.65 sec ≈ 45 sec
actual green time for phase 2, G2= 39.31-3+2.4= 38.71 sec ≈ 39 sec
actual green time for phase 3, G3= 5-3+2.4=4.4 sec ≈ 5 sec
actual green time for phase 4, G4= 26.24-3+2.4 =25.64 sec ≈ 26 sec

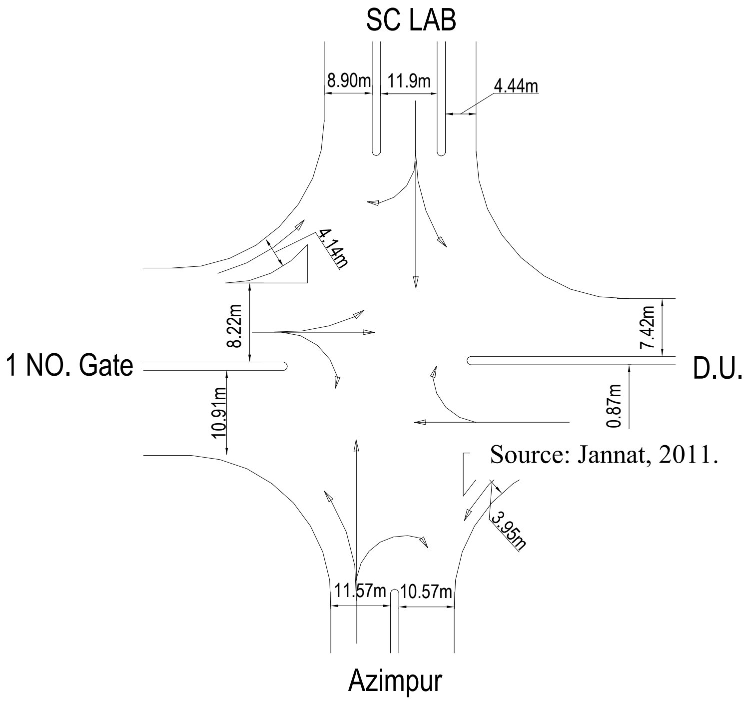
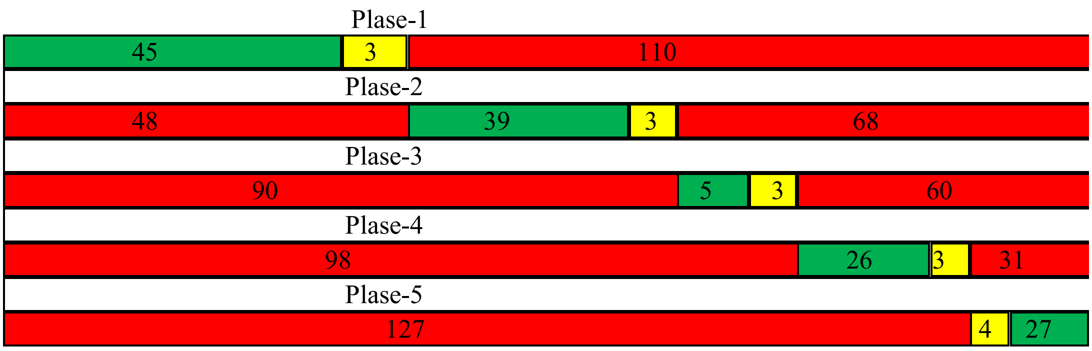


Figure 5: Geometric features of New Market intersection

Pedestrian time can be found out from
G5 =4+32.01/1.2 =30.25 =31 sec

The cycle length =45+5+26+39+31+12
=158 sec

Total 158 sec cycle time



Source: Jannat, 2011.

Figure 6: Phase plan and signal timing for New Market intersection

6. POTENTIALS OF PEDESTRIAN FRIENDLY GREEN STREET

Future road development programs must incorporate pedestrian friendly green street for the sustainability of the city. Green infrastructure is the network of green spaces and water systems that delivers multiple environmental, social and economic values and services to urban communities. Green streets include many enhancements designed to support walking and cycling in an attractive open space environment such as wider boulevards, sidewalks, multi-use pathways, street trees and other landscaping, and roadway features. It replenishes groundwater and serves as green urban connectors of neighbourhood (Munni, 2013).

7. RECOMMENDATIONS

Following are some recommendations provided for holistic thinking and facilitating safe pedestrians and future transportation policy making:

- A common consensus should be built among professionals such as engineers, architects, town planners, bureaucrats, researchers and academicians regarding any policy issue to be formulated and imposed on pedestrian promoted future transport planning.
- Pedestrians should be recognized as a mode of transportation and all the infrastructural facilities should be provided and legal enforcement should be imposed strictly to make the pedestrian ways free from any encroachment.
- Social capacity building is necessary to encourage people to walk and make the footpaths net and clean and free from hawkers.
- Pedestrian friendly green streets should be designed especially for short distance East-West connecting roads of Dhaka City.
- Awareness raising is needed among the pedestrians, vehicle users (owners, drivers and passengers) and all concerned stakeholders (parents of young children, schools, police departments and others) about safe road crossing and pedestrian safety.
- Exclusive Pedestrian Phase may be introduced in some selected intersections on pilot basis to improve pedestrian movements and provide pedestrian-friendly environment. It can be introduced where pedestrian traffic volume is high and crossing is haphazard.
- Visible cross marking must be installed in all the intersections where high density of pedestrian traffic exists. Median island with median barrier also to be provided to ensure safe pedestrian crossing.
- Pedestrian crossing should be considered carefully in planning and designing the intersections and mid blocks. Pedestrian crossing safety should be ensured by improving traffic management, intersections and midblock designs, signal designing and pedestrian facilities such as signs, visible cross marking, grade separated crossing facilities like overpass, underpass etc.
- Improved pedestrian safety at intersections requires coordination among public authorities, professional engineers, media, education experts and vehicle designers to reduce both the number and severity of pedestrian collisions.
- Special pedestrian police or guards may be introduced for prohibition of any type of encroachment.
- Measures should be taken that move in a positive and definitive manner to reclaim the full potential capacity of the existing roads by relocating or removing inappropriate and illegal non-transport related activities from the public right-of-way.
- Vehicles should be regulated properly to reduce speed at the zebra crossing in all the semi-automated signalized intersections.
- In Dhaka, EPP can be implemented through Public Private Partnership (PPP) as huge infrastructural work is necessary. It can be partly funded by the owner of adjacent economic and commercial establishments as they will be benefited from the project.
- Special attention should be given for the design of road by developing guidelines for side mounted, overhead, special crosswalk, pedestrian signals and pedestrian overpasses. The use of signing, marking and signals appropriate for this hierarchical system of pedestrian control should be established.

8. CONCLUSION

There are enormous scopes of promoting pedestrian facilities in the cities and towns of Bangladesh. Many roads that still have enough space can be redesigned and future planned roads may be transformed into green streets which may encourage pedestrian movement and bicycling and sequentially have positive impact in reducing the use of motorized traffic. Green infrastructure contributes pedestrian facilities, environment friendly transportation and ensures healthy urban life. As the country is still developing, it poses opportunities to incorporate any policy for future planning. As the country inherits the image of green, if facilitated it will definitely contribute to develop green heaven future cities in Bangladesh with aesthetics and more picturesque in the world.

9. REFERENCES

Devcon (2009). "Clean Air and Sustainable Environment Preparation Project". Department of Environment, Government of the Peoples Republic of Bangladesh, Devconsultants (Devcon) Limited, Bangladesh.

DTCB (2007). Strategic Transport Plan for Dhaka, Dhaka Transport Coordination Board, Ministry of Communication. Dhaka.

Ha, Seong-Kyu (2013). "The Impacts of Rapid Urbanization and Housing Policy in South Korea". Proceedings of Urbanization: Sustainable Planning and Diversity. Beijing Forum 2013, China.

Hossain, Md. Monjur and Afroze, Mahzabin (2012). "Climate Change Efficacy as An Asian Partner". Proceedings of International Conference on Industrial Waste Management and Process Efficiency. Dhaka University of Engineering and Technology (DUET), 12-14 November, 2012, Gazipur, Dhaka, Bangladesh

Jannat, Hasinae (2011). Pedestrian Road Crossing Behavior at Signalised Intersections and Mid Block Sections in Dhaka City, unpublished MSc Thesis. Department of Civil Engineering, Bangladesh University of Engineering and Technology. Dhaka

Mehndiratta, S. and Deakin, Elizabeth (2010). "How Large Arterials and Land Use Affect Midblock Crossing in Fushun, China". The Journal of Transport and landuse. World Bank, Beijing.

Munni, Sheikh Rubaiya Sultana and Hossain, Md. Sajjad (2013). "Green Street: A Potential Response to the Environmental Aspects of Urbanization of Dhaka City". Proceedings of Urbanization: Sustainable Planning and Diversity. Beijing Forum 2013, China.

RAJUK (1995). Dhaka Metropolitan Development Plan (DMDP), Vol-2: Urban Area Plan (1995-2005), RAJUK (MOHPW), Dhaka.

Waresh, Md. A. (2001). "Effect of Pedestrian Underpasses on Traffic Flow Characteristics of Metropolitan Dhaka". M.Engg Project, Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka.

ANALYSIS OF BUS NETWORK COVERAGE FOR DHAKA CITY ALONG WITH ITS SERVICE QUALITY

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ABSTRACT

Bus is the only mode of mass transit available in Dhaka. It carries about 1.9 million passengers in an average working day out of total about 21 million trips. Share of bus has increased from 10 to 30 percent for all types of trips. There are 11,060 buses and 8,583 mini buses plying on roads which represent only about 3% of total motorized traffic. Out of 160 bus routes 140 routes are operational of which 88% of the routes are being operated by 137 bus companies. The bus routes are delineated in the north-south direction and is restricted to about 200 kilometers only. These routes were never the result of any assessment of demand or proper planning in respect to service optimization and thus resulted in high density route overlapping, undesirable route length and uneven competition among bus companies and not meeting passenger trip patterns. Present bus services lack both standard service as well as door to door service. Reorganization of the bus routes, improvement of bus service quality that will draw consumer's attention, lane separation for MV and NMV's, introduction of BRT and MRT are some of the important measures that need to be initiated in this regard.

Keywords: *Dhaka city, Public transport, Bus network, Route restructuring, Service quality.*

1. INTRODUCTION

Dhaka is one of the only seven cities in the world which has experienced urban population growth higher than 2.4% in between 1975 to 2005 (United Nations, 2006). It has developed into the capital of a nation from a mere provincial capital since the birth of Bangladesh, unfortunately in an unplanned way. It is perhaps the only city in the world without any well and properly planned mass transit system (Hossain, 2006). The environment of Dhaka city transport is characterized by traffic congestion and delays, inadequate traffic management, unaffordable and inaccessible for majority of the people, high accident rates and increasing air pollution problems etc. However, the major problem to this situation seems to be the operational weakness of the present resources. In general, Rapid growth, low incomes, and extreme inequality are among the fundamental reasons of transport problems in Dhaka, similar to every other megacity of developing countries (Pucher et al., 2005). Due to the growing mobility, use of private transport has been increasing and ultimately, it has been a matter of great concern regarding its implications in terms of congestion, pollution and social impact. To reduce this congestion and pollution, public transport in terms of mass transit is preferred over private transport.

2. BUS FLEET AND SHARE IN DHAKA

Dhaka suffers from critical and deteriorated traffic congestion, despite low level of motorization, largely due to the absolute lack of roads, deficient road network configuration and inefficient traffic management (JBIC, 2000). Existing public transport system, bus transit operations in particular is characterized as far short of the desirable mobility needs of the people in terms of reliability, comfort, speed and safety. In Dhaka, buses are generally considered unreliable and time consuming to reach the destination (Hoque et al., 2012).

2.1 Vehicular Growth

Bangladesh has been experiencing rapid vehicular growth in recent years. It has also been observed that the number of motorized vehicle is growing rapidly. Around 300 new vehicles are coming to road every day. In Bangladesh the total number of registered motorized vehicles has become 17,51,834 in June, 2012 increasing from 7,03,215 in 2003 (more than 200% increase in less than 9 years). But astonishing fact is, more than 40% of all registered vehicles are in Dhaka (total 17, 51,834 in Bangladesh) (BRTA, 2012). The trend of motor vehicle growth is shown in Figure 1 below.

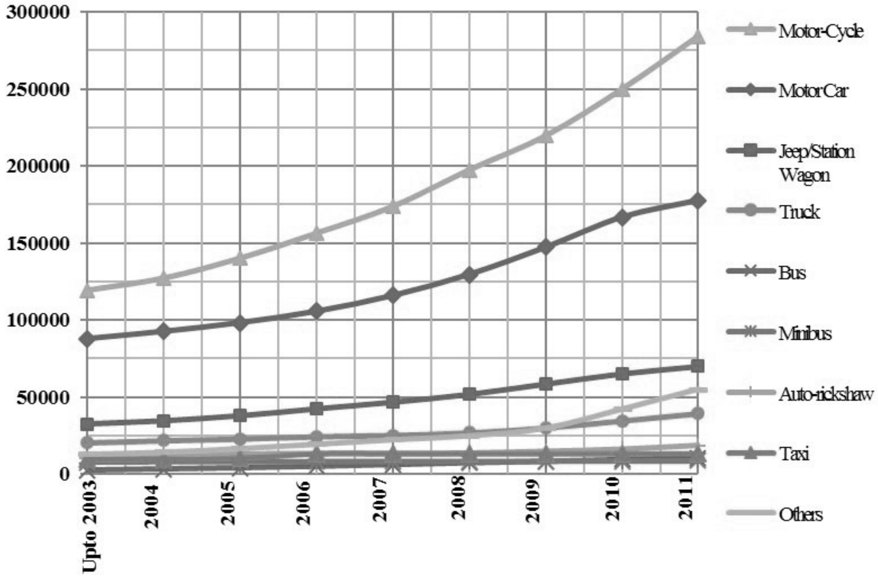

Figure 1: Motorized vehicle growing trend in Dhaka (Hoque et al, 2012)

2.1.1 *Relative Growth of Bus & Minibus in Dhaka*

Despite the demand for public transport service has increased considerably, public transport such as bus and minibuses have not grown substantially. There are 11,060 buses and 8,583 minibuses registered (as of June, 2012) which represent only about 3% (buses and minibuses combined) of total motorized traffic. Though the number of large buses remained nearly constant, the percentage share of bus fleet (buses and minibuses combined) has been in fact declining day by day due to decrease in number of minibuses (see Figure 2). This fact has resulted in further congestion of the roads and worsening air pollution, noise, and safety problems.

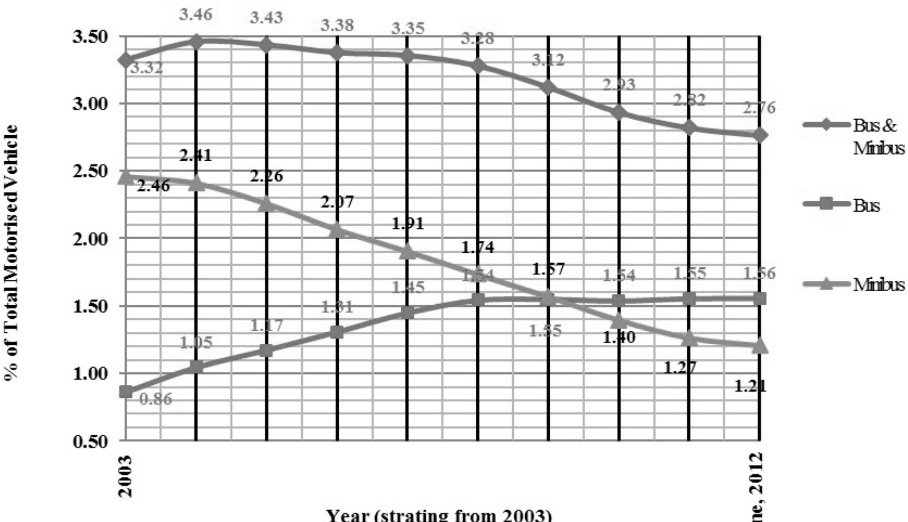

Figure 2: Percent Growth of buses and minibuses in Dhaka (Hoque et al, 2012)

2.2 Modal Share

The prevailing situation is even worse when taken into account the fact that, this inadequate road space is shared by both motorized and non-motorized traffic (heterogeneous traffic mix) and vehicles with varying characteristics (e.g. three-wheelers, human haulers, pickups, vans etc.). On an average day 21 million trips are taking place in Dhaka metropolitan area (ALG, 2011). Despite the rapid growth of motorized traffic in Dhaka, non-motorized transport still remains the dominant mode for the city dwellers who are mostly middle and low income groups. More than 40% of the city trips (see Table 1) are served by walking and rickshaw (DHUTS, 2010). The varied traffic mix and heavy concentration of non-motorized vehicles with almost 70% of the available road space is occupied by rickshaws and their dominance is expected continue in the foreseeable future (Hoque and Hossain, 2004). Currently, rickshaw movements are however restricted in some major roads.

Table 1: Modal share in metro Dhaka

Mode	Percentage of Share				
	DITS (1994)	DUTP (1997)	JBIC Study (1999)	STP (2005)	JICA Study (2009)
Walk	60.1	62.82	62.05	14.0	19.09
Rickshaw	20.1	20.04	13.28	34.0	38.19
Bus	12.8*	10.42*	10.22	44.0*	29.83
Passenger Car	7.0**	6.72**	3.97	8.0**	4.30
Others			4.65		2.86
Total	**100**	**100**	**100**	**100**	**100**

* Transit
** Motorized (Non Transit)

The modal distribution by income groups is shown in Table 2. It shows that trips on foot is made by the low income group (73%) while most of the rickshaw trips are made by the middle income group (59%) (DHUTS, 2010). The significance of walk and rickshaw trips is clearly evident as they relate to 97% of the city dwellers. The following Table 2 shows modal distribution (in terms of trips) by income groups. From the table, it is clear that the low income group is responsible for the lion's share of trips on foot (73%) while most of the rickshaw trips are made by the middle income group (59%) (DHUTS, 2010).

Table 2: Modal share of trips with respect to income groups

Income Group	Proportion of Income Groups (%)	Modal Share			
		Walk	Rickshaw	Transit	Motorized (non-transit)
Low (<12,500)	48	73	38	41	14
Medium (12,500-55,000)	49	26	59	56	66
High (>55,000)	3	1	3	3	20
Total	**100**	**100**	**100**	**100**	**100**

2.3 Extent of Road Network

In an ideal city, 25% of the surface area should be used for constructing roads and lanes, (Hossain, 2006) but Dhaka has only 8% (DCC, 2002).

Table 3: Road network within Dhaka City Corporation (DCC) area

Type of Road	Definition	Length (km)	Percentage (%)
Primary	Minimum of 31m wide, 6 lanes of 3.25m, footpath and median	68.45	5.29
Secondary	Minimum of 25.50m wide, 4 lanes of 25m, 2 NMV lanes of 2m wide, footpath and median	108.20	8.37
Connector	Minimum of 22 m wide, 2 lanes of 3.25m each, 2 NMV lanes of 2m each, footpath and median	221.35	17.12
Local	Minimum of 8.75 m wide, two lanes of 1.36m and footpath (1.5m each way)	573.75	44.37
Narrow	Minimum of 4.5 m wide with two lanes of 1.36m	321.27	24.85
	Total	**1293.02**	**100**

Moreover, like most of the developing cities, Dhaka's road network hierarchy is poorly defined, with very limited number of arterial and main roads. It can be observed that by considering connector roads suitable for bus flow, there is around 30% road available for bus services, which is only about 400 km (see Table 3). Moreover, this road space is also shared by NMTs and as a result for absence of any bus priority measures, buses often come second when they compete for road space with other modes.

3. STATUS OF THE PRESENT BUS NETWORK

3.1 Bus Route Layout and Distribution

As per the records of the Bangladesh Road Transport Authority (BRTA), the number of bus routes in Dhaka is 160. These routes go mainly through high capacity arteries due to the functional characteristics of the road network. Only 12.5% of the entire road network is a motorway or main street, suitable for bus services (DTCA, 2012).There is a number of satellite townships like Savar, Narayanganj, Gazipur, Narsingdi, Demra etc. which are connected to Dhaka by sub urban bus service mostly minibuses. Within the city area, these sub urban buses pick and drop passenger on the city road and illegally operates up to the city centre. Therefore, within the city areas these sub urban buses operate as city buses.

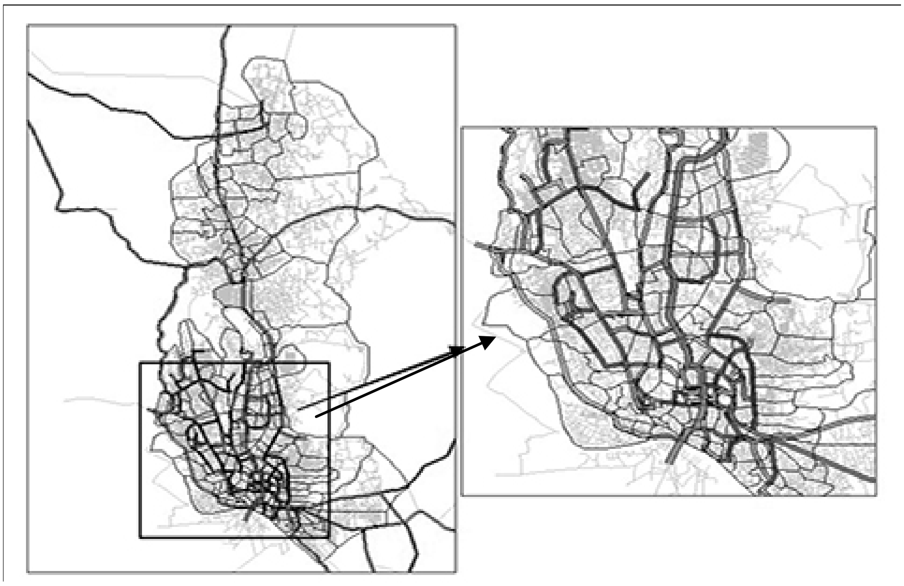

Figure 3: Current Bus Network

Most of the main arterial roads of Dhaka are towards north-south and there are serious deficiencies of roads from east to west. As a result, most of the bus routes of Dhaka City are also north-south. In fact, just 5 out of the 152 bus routes run east-west (DTCA 2011). Areas close to the riverside, which were the first to be developed, such as Old Dhaka and Kamrangirchar, have almost no bus supply due to the urban characteristics. High density of buildings is found without following any specific pattern amongst narrow streets. In general, lack of bus routes is found either in dense area with narrow streets or in areas with water bodies or without road connections. The distribution of routes by length is shown in Table 5. It shows that nearly 70% of the route length is within 11 to 30 km.

Table 4: Distribution of routes by length

Route Length (km)	Number of Routes	Percentage (%)
<10	1	0.97
11-20	39	37.86
21-30	31	30.10
31-40	17	16.50
41-50	5	4.85
51-60	7	6.80
>71	3	2.91
Total	**103**	**100.00**

3.2 Bus Trip Production

The total number of trips by bus on an average working day is 70, 84,159. Mobility within DMA represents 89% of the total mobility by bus (79% in DCC) and those inside Gazipur district only represent 5.3%. The rest are flows between both areas. According to the public transport model zoning (168 zones) and 4.5% of those are intrazonal, with the origin and destination inside the same zone. One third of the total trip production by bus is concentrated in Mirpur, Pallabi, Kafrul, Dhanmondi and Mohammadpur. Ramna itself is the highest trip producer, generating and attracting 12% of the total trips followed by Motijheel with 9% of total production.

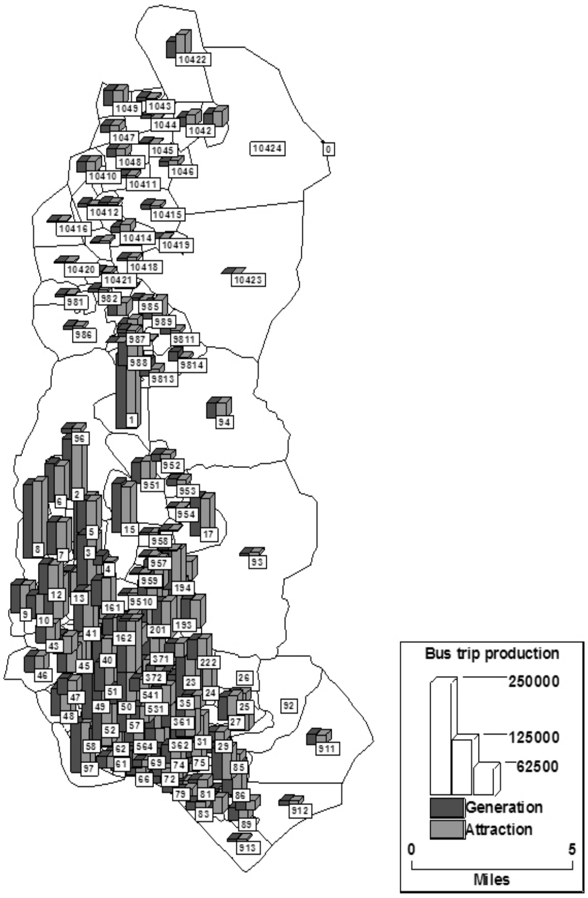

Figure 4: Bus trip production on an average working day

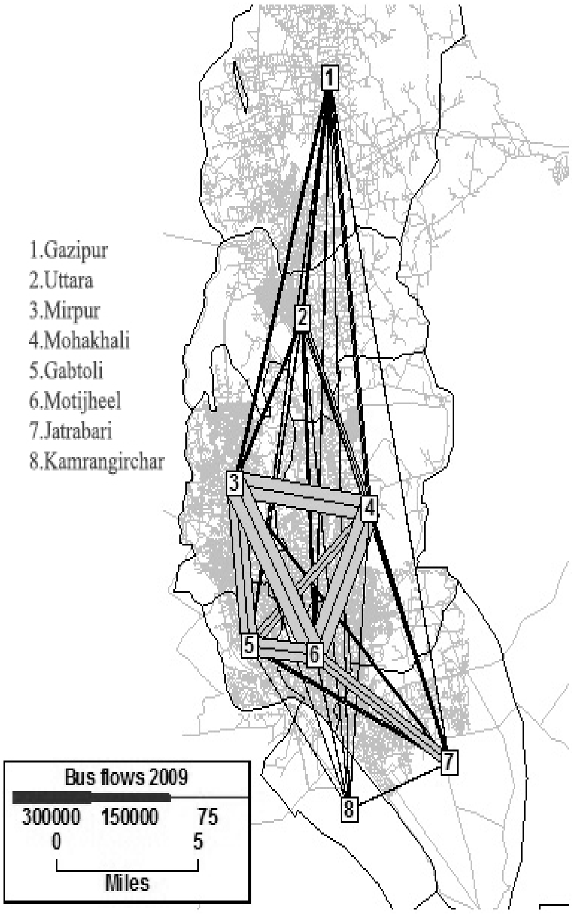

Figure 5: Daily bus flows between zones

From this exercise it can be stated that 75% of the total trip production by bus is generated and attracted within Central Dhaka (zones 3, 4, 5 and 6). Gazipur and Uttara produce 15% of the total trip production by bus in the study area and 45% of which stay within these zones (1 and 2). Excessive Route overlapping is given in Table 5

Table 5: Top Seven locations of overlapping routes

Location	Number of Routes overlapping
Paltan	56
GPO	50
Motijheel	59
Shahbagh	48
Press Club	43
Gulistan	41
Airport	36

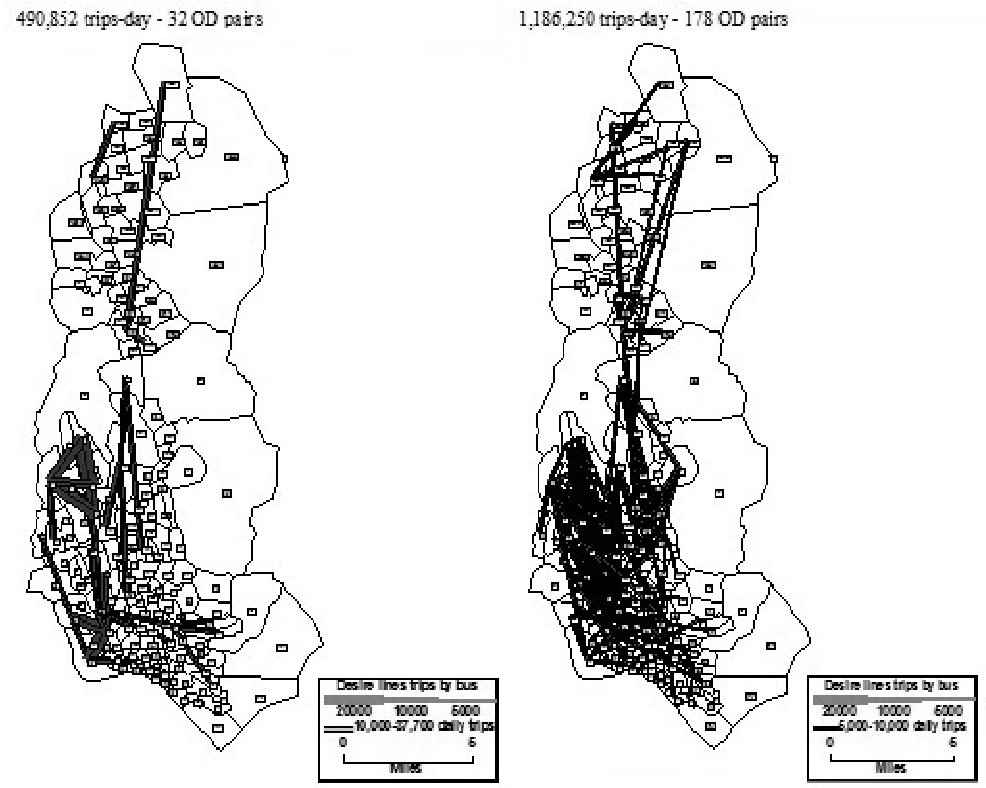

Figure 6: Trip flows by bus greater than 5,000 trips/days

The average length of trips by bus is 5.8 km. Flows over 10,000 daily trips have an average trip length of 3.9 km and represent 4% of the total intrazonal trips, and flows from 5,000 to 10,000 have an average trip length of 5.1 km and represent 9.1% of total intrazonal trips. Longer trips are those with trip flows less than 1,000 daily trips (average length of 7.6 km). Intrazonal trips by bus represent 4.5% of the total number of trips, as bus is used mainly for long trips between zones. The zones with the highest percentages of intrazonal trips (>6.5%) are generally big zones or peripheral zones, especially in Gazipur Upazilla, as the figure below indicates.

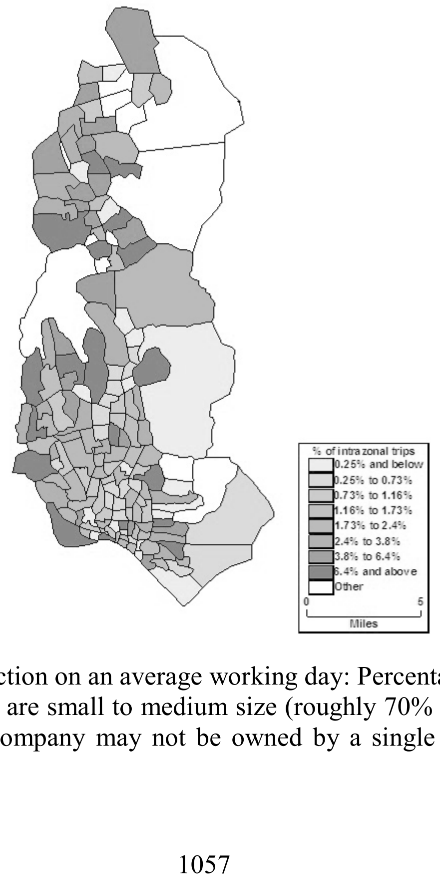

Figure 7: Bus trip production on an average working day: Percentage of intrazonal trips

Most of the bus companies in Dhaka are small to medium size (roughly 70% of them have 11 to 30 buses) (see Table 6). However, all buses of a company may not be owned by a single person, rather a good number of individuals own one or more buses, make a group, form and run a bus company. So, even these companies' buses ply on the roads like individually owned buses.

Table 6: Distribution of Bus Companies by Fleet Size

Fleet Size	Number of Companies	Percentage (%)
1-10	2	2.67
11-20	21	28.00
21-30	31	41.33
31-50	14	18.67
51-70	4	5.33
71-99	1	1.33
100-150	1	1.33
>150	1	1.33
Total	**75**	**100.00**

The current bus network system covers 5,325,441 inhabitants and 141.43 sq.km of the study area (DMA and Gazipur Upazilla), considering the coverage as the area and population within a buffer of 400 m of the transport network (population is that of 2009). It covers areas with high population density, being the average density of the areas covered 54,011 inhabitants per km^2.

Figure 8: Bus network coverage (band widths of 400m)

4.

5. EXISTING BUS SERVICE SCENARIO

5.1 Bus Frequency

Dhaka has five major bus routes terminating/originating locations which can be seen in the table below. These routes are mainly part of north south corridors, from Gazipur, Savar, Naraynganj, Tongi, Uttara and Mirpur sub urban areas and ending mostly at the city centre at Motijheel/Gulistan/Fulbaria/Sadarghat and some are extended up to the south eastern fringe of the city.

Table 7: Key statistics of bus minibuses routes

Origin/Destination locations	Number of Routes	Number of Buses	Number of Minibuses	Total
Mirpur, Pallabi, Gabtoli, Kulsi	38	537	723	1,136
Mohammadpur, Shaymoli, Adabar,	13	252	182	434
Manikganj, Dhamrai, Hemayetpur, Savar, EPZ, Chandra, Konabari, Kaliakur	30	659	528	1,187
Joydebpur, Boardbazar, Kapashia, Tongi, Kaliganj, Ashulia, Kuril, Gulshan,	36	579	1,219	1,798
Naraynganj, Meghnaghat, Sonargaon, Ghorashal, Katchpur Bridge, Adamjee	16	178	265	443

As displayed in the following figure, bus routes are highly concentrated in high capacity arteries. The average bus supply in a motorway link is 2,384 buses/day/direction, which means that there are 2.5 buses every minute. In main streets there are 2.15 bus/min and in local streets 0.67 bus/min on average (DTCA, 2012).

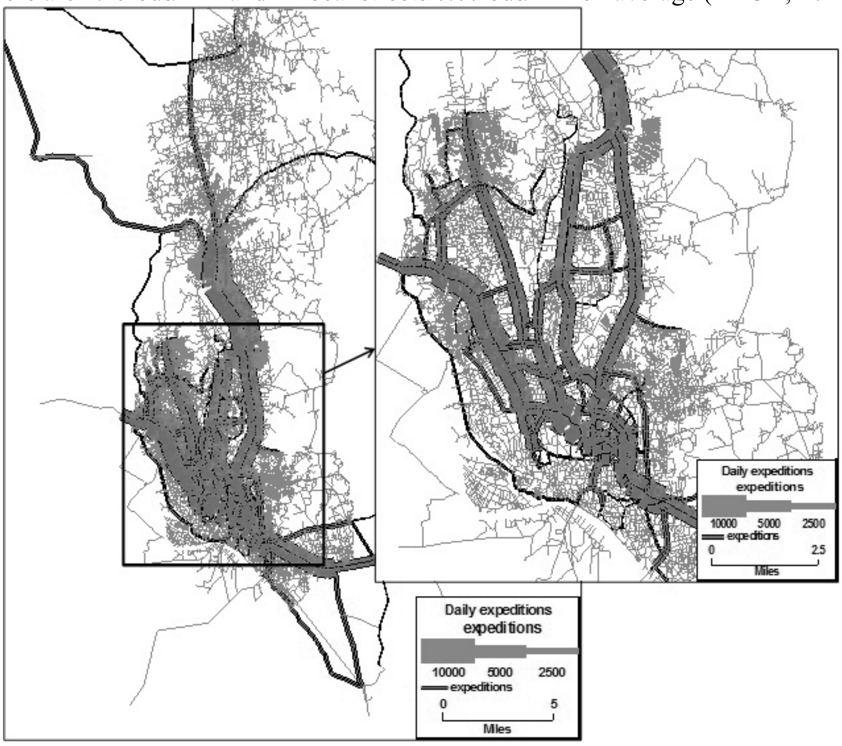

Figure 9: Daily frequency of bus services on the road sections of Dhaka

According to these estimates, bus supply in an average working day reaches 9,500 bus/day/direction in some roads in DMA (which means almost 10 buses per minute in one direction across that road section), although doesn't exceed 1,150 bus/day/direction outside its boundaries. In the north, N3 from Gazipur to Tongi offers 1,143 bus/day/direction and 4,497 bus/day /direction from Tongi to Uttara. There are 908 bus/day/direction along N302 and 276 bus/day/direction along R301. In south-east of DMA, there are 174 daily expeditions that run along N2 and 192 along N1 (DTCA, 2012).

In DMA, N3 (Airport Rd) reaches values of 6,700 bus/h/direction between N501 and Kuril, and in N5 up to 7,000 bus/day/direction. The highest values are found in Dhaka city centre, with 9,500 bus/day/direction in Nilkhet road, as most of the bus routes have their origin and destination in Paltan or they cross the city center. Bus supply is also high along N1 with 6,000 bus/h/direction (DTCA, 2012).

5.2 Service Quality

Service quality is a measure of how well the service level delivered matches customer expectations, while a firm delivering quality means conforming to customer expectations on a consistent basis.(Jeowono and Kubota, 2007). Bus services available in Dhaka city can be categorized in two groups: counter bus service and local bus service. Counter bus service has specified stoppages for boarding and alighting of passengers and the tickets are sold at the counter of those stoppages. For such buses, passengers have to purchase their tickets from the bus counters just before boarding into bus. A very small number of counter buses are air-conditioned. In contrast, local bus service has no specified stoppage (stop anywhere on the way for boarding and alighting passengers) and passengers pay the fair to the bus conductor on bus (after boarding). Buses of the both types often remain heavily over-crowded; mostly because of a gap between demand and supply. However, there are a few seating service buses operating in certain limited routes which allow boarding passengers only if there is an empty seat available for the person. These buses also boarding and alighting passengers at the specified stops (sometimes allow passengers to alight at any place where the passengers want to), and the passengers have to pay for their tickets inside the bus. Only 6 seats in each bus have been reserved for the female passengers but nothing such for disabled people or senior citizens (Rahman, 2010).

The condition of the buses plying in Dhaka city is not good. This is because the majority of bus fleet is very old and the maintenance is almost absent or very poor. The physical conditions of the buses are not satisfactory and 25% of the buses are in worse condition. Despite the poor bus condition users are not feeling it because their main concern is getting a seat or room inside the bus irrespective to its overall condition. Near about 38% of buses have poor interior condition (Rahman, M. and Nahrin, K. 2012).

Although BRTC has started with the banner of **"***Service is our motto: Comfort is our commitment",* it cannot fulfill any of their objectives. With few exceptions, the bus owners or operators, including government owned BRTC buses, in Dhaka City do not pay adequate attention to passengers' comfort (Andaleeb *et al*. 2007). Internal facilities, such as lights and fans, are frequently out of order or in need of repair. Many of these buses do not have fans and lights at all. Other amenities, such as lighting during night, airflow and ventilation are not constantly monitored to ensure a desirable passenger experience. Basic passenger requirements, like comfortable seats and windows for airflow, also do not measure up to the standards.
There are many concerns related to the current bus service as per the information and the feedback obtained by different studies and surveys, among them are (DTCA, 2011):

- Long waiting time
- Non availability of a seat and over-crowding
- Inefficient bus fare and ticketing system
- Unfavorable condition inside the bus
- Long delay time
- Physical harassments of the female passengers
- Misbehavior of the bus staff
- location of the bus station and accessibility
- Poor facilities of the bus stops
- Non availability of information about bus services

Despite the crucial function of public buses plying in developing countries, their roles are disrupted for different reasons. Transportation system of Dhaka city is under huge challenge of managing growing number of private cars and non motorized vehicles. Traffic congestion had become an everyday scenario of this city. Policy maker consider increasing the number of public buses as solution in most cases. But increasing the number of buses is not only the solution of present problem of Dhaka city. Bus service quality has to be improved to meet the expectation of customers which can influence them to use public transport (Bus) than private cars.

Figure 10: Overcrowding Condition

6. CONCLUDING REMARKS

There is an ever increasing urgency of mitigating the complex transport problems in Dhaka. Poor traffic management, lack of road spaces and fragmented bus network resulted in severe traffic congestion, massive delays, increased fuel wastage and resource losses. Certainly Dhaka City needs better public transportation system e.g. BRT and MRT and reorganization of bus network by eliminating the overlapping routes. Few minor initiative in bus service such as advanced ticketing system, discipline among the passengers while waiting at bus stoppage or boarding and alighting buses, making all buses as sitting service and reserve few seats for the disabled, etc could bring better results for bus service. Provision of bus-only lane, increasing the number of bus fleet, maintaining a specified time schedule and publicize the information widely to make it available for the passengers, adjusting the fare rate rationale to the service level, ensuring better air circulation inside the vehicle, reducing the delay time in station, etc. Thus, the overall condition of the existing bus network and its coverage as well as the aspects of service quality is briefly discussed here.

7. REFERENCES

Bangladesh Road Transport Authority (BRTA) (2012), 'Number of Yearwise Registered Motor Vehicles in Dhaka', Available at: http://www.brta.gov.bd/images/files/motor_v _dhaka_05-08-12.pdf.

Bhuiyan, A.A. (2007), 'Study on Bus Operation in Dhaka City', 'Air Quality Management Project (AQMP)', Final Report, Department of Environment (DoE), Ministry of Environment and Forest (MoEF).

DITS (1994), Greater Dhaka Metropolitan Area Integrated Transport Study, Planning Commission and Department of Economic and Social Development, January 1994.

Dhaka Transport Co-Ordination Board (DTCB) (1997), 'Dhaka Urban Transport Project (DUTP)', Final Report.

DevConsultants Limited Bangladesh (DevCon) (2009), 'Consultancy Services for Pilot Bus Priority Corridor Pre- Feasibility Study', 'Clean Air and Sustainable Environment (CASE) Preparation project', Final Report, Department of Environment (DoE), Ministry of Environment and Forest (MoEF).

Dhaka Transport Co-ordination Authority (DTCA) and Advanced Logistics Group (ALG) (2012), "Dhaka Bus Network and Regulatory Reform Implementation Study and Design Work", Contract for Consultants Services between DTCA and ALG.

Dhaka Transport Co-ordination Authority (DTCA) and Advanced Logistics Group (ALG) (2011), "BRT and Corridor Restructuring Implementation Study and Preliminary Design Work for the Uttara–Mohakhali–Ramna Sadar Ghat Corridor in Dhaka", Contract for Consultants Services between DTCA and ALG.

Dhaka Transport Co-Ordination Authority (DTCB) (2010), 'Dhaka Urban Transport Development Study (DHUTS)', Final Report, Bangladesh University of Engineering and Technology (BUET) and Japan International Cooperation Agency (JICA) Study Team.

Dhaka City Corporation (DCC) (2002), Structure plan, master plan and detailed area plan for Dhaka city, Volume. 1, Dhaka, Bangladesh.

Hoque, M.M., Barua, S., Ahsan, H.M., and Alam, D. (2012), 'BRT in Metro Dhaka: Towards Achieving a Sustainable Urban Public Transport System', Proceedings of CODATU XV: The Role of Urban Mobility in (re)shaping Cities, Addis Ababa, Ethiopia.

Hossain, M. (2006), 'The Issues and Realities of BRT Planning Initiating in Developing Cities', Journal of Public Transportation, 2006 BRT Edition.

Japan Bank for International Co-Operation (JBIC) (2000), 'Study of the Improvement of Transportation and Environment in Dhaka', Dhaka, Bangladesh.

Jeowono, T.B., Kubota, H. (2007), User satisfaction with paratransit in competition with motorization in Indonesia anticipation of future implications. Transportation 34, 337-354.

Pucher, J, Korattyswaropam, N., Mittal, N. and Ittyerah, N. (2005), 'Urban transport crisis in India', Journal of the 'World Conference on Transport Research Society', Volume. 12, Issue. 3.

Rahman, M.S.U. 2010, Bus service for 'Women Only' in Dhaka City: an investigation, Journal of Bangladesh Institute of Planners (JBIP), 3, pp.17-32.

Rahman, M. and Nahrin, K. (2012), "Bus Services in Dhaka City – Users Experiences and Opinions" Journal of Bangladesh Institute of Planners Vol. 5, December 2012, pp. 93-105, @ Bangladesh Institute of Planners, ISSN 2075-9363.

STP (2005), "Strategic Transport Plan for Dhaka", Draft Report, Prepared by Louis Berger Group and Bangladesh Consultant Ltd.

United Nations (2006), 'World Urbanization Prospects: The 2005 Revision', Working Paper No. ESA/P/WP/200, Department of Economic and Social Affairs, Population Division.

Wright, L. (2003), 'Bus Rapid Transit, Module 3b, Sustainable Transport: A Sourcebook for Policy-makers in Developing Cities', Deutsche Gesellschaft für Technische Zusammenarbeit (GTZ) Transport and Mobility Group.

INCREASING AIR TRAFFIC DEMAND AND RELEVANT ISSUES IN BANGLADESH

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ABSTRACT

Over the last century aviation has been a catalyst for economic growth and has enriched the quality of life for people all over the world. Air travel demand in Bangladesh is also increasing rapidly along with the increasing economy. Passenger carrier has registered an average 8% growth in the last 3 years. In cargo sector, the growth figure is phenomenal at 13%. It is expected that an increase in passenger volume from 5.8 to 10.2 million and increase in air cargo from 2, 30,000 to 3, 60,000 metric tons will take place by 2020 in Bangladesh. To meet the growing traffic demand at present Biman Bangladesh has included several wide bodied modern aircrafts in the fleet. At present the aviation activities are carried out from three international and five domestic airports and about seventeen airlines are now operating in and out of the country. Provision for increased air travel demand and accommodating new aircraft types demand airport configurations as well as airport runways need to be upgraded to world-class standards according to the design standards of FAA and ICAO. This paper presents some guidelines for future improvement options to meet the future air travel demand.

Keywords: Increasing air traffic, Regional hub, New generation aircrafts, Airport configurations, Airfield pavement.

1. INTRODUCTION

Airports are unique entities that have profound economic, social, and environmental effects on a local, regional, and even national level. They provide the means for the efficient movement of passengers and goods to virtually anywhere in the world, playing a vital role in the trend toward “globalization” and the inter-connections between international trade and local economies (Ashford, 2011). Airports are shaping urban space in the twenty-first century much as highways did in the twentieth century, railroads did in the nineteenth century, and seaports did in the eighteenth century.

Bangladesh is a developing country and its economy is increasing significantly as well as the air travel demand. In 2011, the number of registered carrier departures worldwide from Bangladesh was 22,005 and the quantity for air freight in Bangladesh was 120.12 million ton-km (Indexmundi, 2013). To meet the demand, at present aviation activities are carried out from three international and five domestic airports and (CAAB, 2013). Hazrat Shahjalal International Airport, Dhaka handles almost 66% of Bangladesh’s air traffic whereas about 21% is handled by Shah Amanat International Airport, Chittagong nearly 4% by Osmani International Airport, Sylhet and the remaining 9% by Bangladesh's five domestic airports (Wikipedia, 2013). But the existing facilities are not sufficient to meet the future air traffic demand of the country.

2. AIR TRAVEL DEMAND IN BANGLADESH

Air travel demand is the critical and fundamental data in the airport planning process. The International Civil Aviation Organization (ICAO) has been compiling statistics on air travel since the start of commercial air travel (ICAO, 2010). They represent three basic descriptors of demand since air travel data started to be globally recorded- Passengers, Aircraft movements and Passenger-distance traveled.

Bangladesh is a small country where the distances between the major cities are not large enough for air transportation to be very effective. In general, roadway is highly congested and accident-prone and railway and waterway are inefficient and slow (Alam, 1998). But the less improvement of roads and the waterways as well as imbalanced socio-political conditions of the country people now a day interested by air travel. So air travel demand is increasing rapidly in Bangladesh (Euromonitor International, 2013).

Bangladesh has a strategic position in Asia Pacific whereby businesses from Pakistan and India see great potential. With cheap labor and a fairly secure environment, Bangladesh has become a low-cost manufacturing hub and many businesses from neighboring countries have even relocated to Bangladesh. India was the leading source market for arrivals in Bangladesh in 2012 and the primary reason for this are the trade links between the two countries (Euromonitor International, 2013)

Bangladesh as a tourist destination has benefited from increased arrivals from India. This is mainly due to its close proximity to the country. Sustained growth over the forecast period will be possible if there is a clear agenda for the Bangladesh Parjatan Corporation (BPC), which acts as the tourism board for the country. Improved infrastructure will not only encourage a greater influx of tourists, but is also likely to lead to a greater possibility of online sales of the various categories within travel and tourism (Alam, 2001).

Air travel demand has experienced very fast growth in the last two decades. The volume of air passenger and freight is expected to increase much faster in near future with increased industrialization and economic development. Congestion at airport runways, passenger, and cargo terminals, inspection and ground transport facilities is expected to increase, as will demand for additional and improved airport system.
The trends of the air travel demand for the last 38 years are shown in the Figures 1, 2 and 3 below (Indexmundi, 2013). Figure 1 shows registered carrier departures worldwide in Bangladesh with a maximum value of 22,005 in 2011 and a minimum value of 5,900 in 1999. Figure 2 shows passengers carried a maximum value of 2,487,382 in 2011 and a minimum value of 431,400 in 1974. Similarly Figure 3 freight (million ton-km) movement with reached a maximum value of 199.80 in 1997 and a minimum value of 0.01 in 2009.

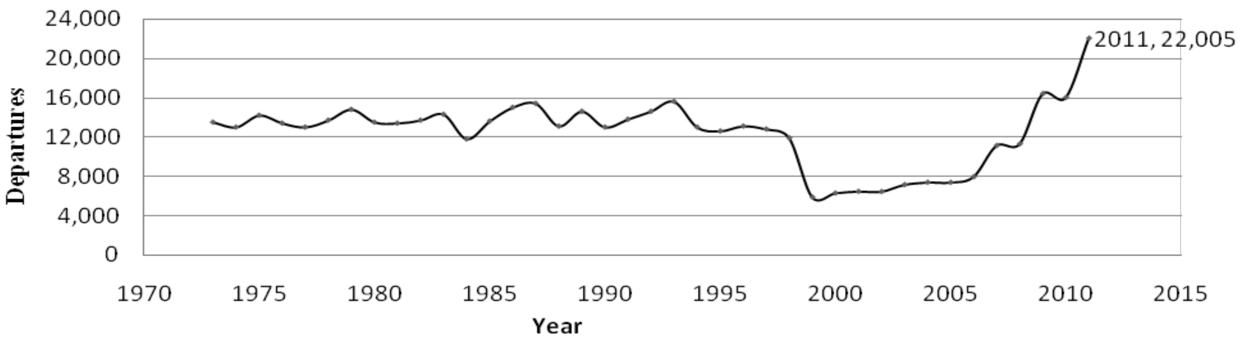

Figure 1: Registered carrier departures worldwide in Bangladesh

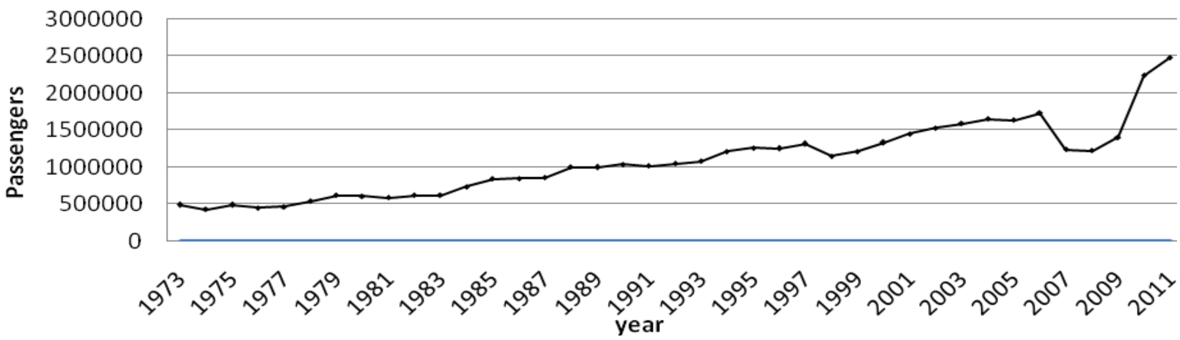

Figure 2: Air passengers carried in Bangladesh

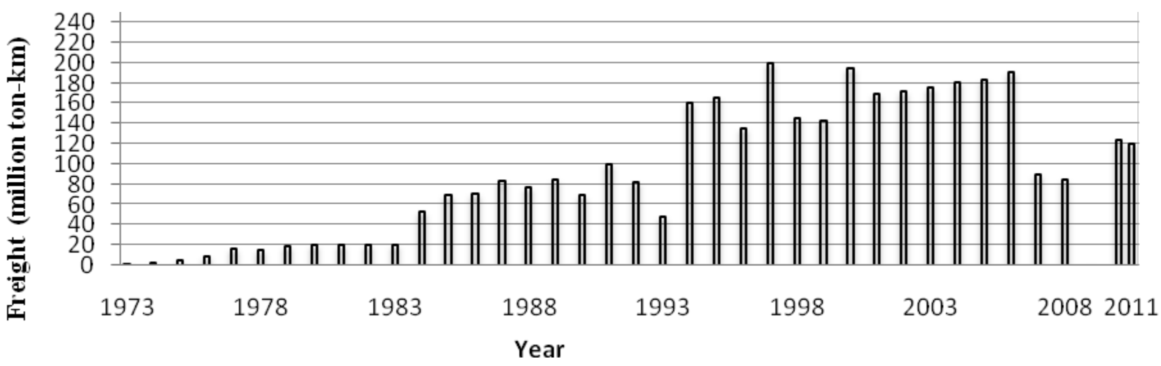

Figure 3: Air transport, freight (million ton-km)

Further weekly passenger service for different origin and destination airports are listed in Table 1, 2 and 3 below.

Table 1: Bangladesh Domestic Air Services (non-stop weekly departures)

Rank	Airport	Weekly Flights	Weekly Seats	% Capacity
1	Hazrat Shahjalal International Airport, Dhaka (DAC)	145	11,668	45.3 %
2	Shah Amanat International Airport, Chittagong (CGP)	78	7,520	29.2 %
3	Osmani International Airport, Sylhet (ZYL)	29	4,128	16.0 %
4	Jessore Airport (JSR)	28	1,218	4.7 %
5	Cox's Bazar Airport (CXB)	14	826	3.2 %
6	Saidpur Airport (SPD)	7	259	1.0 %
7	Rajshahi Airport (RJH)	4	148	0.6 %
8	Others	-	-	-
	TOTAL	305	25,767	-

Table 2: Home Airport: Bangladesh International Air Services (Non-Stop Weekly Departures)

Rank	Airport	Weekly Flights	Weekly Seats	% Capacity
1	Hazrat Shahjalal International Airport, Dhaka (DAC)	256	57,514	89.8 %
2	Shah Amanat International Airport, Chittagong (CGP)	38	6,514	18.8 %
3	Others	-	-	2.1 %
	TOTAL	294	64,028	-

Table 3: Destination Airport: Bangladesh International Air Services (Non-Stop Weekly Departures)

Rank	Airport	Weekly Flights	Weekly Seats	% Capacity
1	Dubai International (DXB)	42	10,552	16.5 %
2	Kuala Lumpur International (KUL)	20	5,521	8.6 %
3	Abu Dhabi International (AUH)	12	4,237	6.6 %
4	Singapore Changi International (SIN)	20	4,227	6.6 %
5	Bangkok Suvarnabhumi International (BKK)	21	3,993	6.2 %
6	Doha International (DOH)	13	3,669	5.7 %
7	Kolkata NetajiSubhas Chandra (CCU)	27	3,540	5.5 %
8	Sharjah International (SHJ)	21	3,402	5.3 %
9	Kuwait International (KWI)	11	2,936	4.6 %
10	Muscat International (MCT)	17	2,918	4.6 %
11	Others	90	19,033	29.7 %
	TOTAL	294	64,028	-

3. AIR TRANSPORTATION SERVICES IN BANGLADESH

3.1 Status of Airport in Bangladesh

The air transportation network in Bangladesh is comprised of three international, seven domestic, five STOL (short Take-off and Landing) ports and nine other unused airports. All airports of the country are, however, in the public sector, only a minor element of the ground facility is in the private sector. The status of airports is listed in Table 4.

Table 4: Airports in Bangladesh.

International airports	Domestic Airports	STOL ports and Unused airports
1. Hazrat Shahjalal International Airport, Dhaka (HSIA) **2.** Shah Amanat International Airport, Chittagong (SAIA) **3.** Osmani International Airport, Sylhet (OIA)	Operational: 1. Saidpur Airport 2. Shah Makhdum Airport, Rajshahi 3. Jessore Airport 4. Barisal Airport 5. Cox's Bazar Airport Non operational: 6. Tejgaon Airport and 7. Ishurdi Airport Under construction 1. Khan Jahan Ali Airport, Bagerhat 2. NoakjaliAirport	1. Comilla STOL port 2. Bogra STOL port 3. Thakurgaon STOL port 4. Lalmonirhat STOL port 5. Shamshernagar STOL port Unused airports: 1. Sandwip Airport 2. Chakaria Airport 3. Feni Airport 4. Rajendrapur Airport 5. Maulvi Bazar Airport 6. Rasulpur Airport 7. Sirajganj Airport 8. Bajitpur Airport 9. PaharKanchanpur Airport

3.2 International Airports and Facilities

Hazrat Shahjalal International Airport is the largest and main airport in Bangladesh located at Uttara, 13 miles (20 kilometers) from Dhaka city. The airport is a primary air travel gateway into Bangladesh and the hub of Biman Bangladesh Airlines, the country's national airline. Its strategic geographical location makes the airport an ideal transfer point for leisure and business passengers travelling around Bangladesh. The airport is well equipped with terminal buildings, hangers, sophisticated radar, a freight village (warehouse) and other modern equipment for handling aircraft. It connects almost all the major cities of the world with Bangladesh. From here, Biman Bangladesh goes to 31 cities on 4 continents.

At Chittagong the second greatest airport named Shah Amanat International Airport has been developed with the financial assistance of Japanese Government. It is now possible to operate wide-bodied aircrafts like Boeing 474-400 there and is furnished with modern facilities for passengers and communication and navigation systems. It is now able to work as an alternate airport to HSIA in case of accidental closer of HSIA and for technical diversions.

Osmani International Airport is located 5 miles north-east of Sylhet serving as the country's third International Airport. Biman Bangladesh Airlines at one point earned most of its revenue from this airport. GMG Airlines, the first private airline in Bangladesh, United Airways (Bangladesh) and Regent Airways also use this airport. The vast majority of passengers using the airport are expatriate Bangladeshis and their descendants from the Sylhet Division living in the United Kingdom.

The physical characteristics and the existing facilities of the airports are shown in Table 5.

Table 5: The physical characteristics and the existing facilities of the airports

Name of the options	Hazrat Shahjalal International Airport, Dhaka	Shah Amanat International Airport, Chittagong	Osmani International Airport, Sylhet
Physical characteristics			
Code	VGZR (ICAO) DAC (IATA)	VGEG (ICAO) CGP (IATA)	VGSY (ICAO) ZYL (IATA)
Land area	1,981 acres (802 ha)	-	-
Latitude	23.843(N)	22.250(N)	24.963(N)
Longitude	90.398(E)	91.813(E)	91.867(E)
Aerodrome Elevation	27ft (8m)	14ft	50ft
Magnetic Variation	-	-	50' West
Temperature	35°C.	32°C	35.4°C.
Existing facilities			
Cargo handling facilities	Fully available	Not fully equipped	Not fully equipped
Hotels	Nil	Nil	Nil
Restaurant Accommodation	Nil	Nil	Nil
Fire Figurehting Services	Available	available	Available

3.3 Airlines in Bangladesh

In total eighteen domestic and nineteen foreign airlines operate in Bangladesh. Among the foreign airlines only Al-Italia has been rendering freighter services. Of Bangladeshi airlines Bismillah airlines operates freighter services between Dhaka and Bangkok. The domestic and foreign airlines are listed in Table 6.

Table 6: List of Domestic and Foreign Airlines of Bangladesh (CAAB, 2013)

Sl No	Domestic Airlines	Foreign Airlines
1	Aero Technologies Ltd.	Air India Express Ltd.
2	Arirang Aviation Ltd.	Air Asia
3	Biman Bangladesh Airlines Ltd.	Bangkok Airways
4	Bangla International Airlines Ltd.	China Southern Airlines
5	Bismillah Airlines Ltd.	Draongair(Cathy Pacific)
6	Bangladesh Flying Academy &General Aviation Ltd.	Druk Air
7	Easy Fly Express Ltd.	Emirates
8	Galaxy Flying Academy Ltd.	Flydubai
9	Meghna Aviation Ltd.	Gulf Air
10	Novo Air	Jet Airways
11	Pacific Aviation Ltd.	Kingfisher Airlines Ltd.
12	Regent Airways (HG Aviation)	Kuwait Airways
13	South Asian Airlines Ltd.	Malaysian Airways
14	Sky Capital Airlines Ltd.	Mihin Lanka
15	Square Air Limited	PIA
16	United Airways (BD Ltd.)	Qatar Airways
17	Tac Aviation Ltd	Saudi Arabian Airways
18	R&R aviation	Singapore Airways
19		Thai Airways

4. AIRPORT CONFIGURATION

The components of an airport are typically placed into two categories i.e. airside and landside components. The airside of an airport is planned and managed to accommodate the movement of aircraft around the airport as well as to and from the air, which includes runways, taxiways, ramp entrance, navigation aids equipment etc.

The landside components of an airport are planned and managed to accommodate the movement of ground-based vehicles, passengers, and cargo. These components are terminal buildings, hangers, aprons, ground lighting etc (Robert, 1994).

4.1 Runway Characteristics

Runway is the single most important facility on the airfield. Without a properly planned and managed runway, desired aircraft would be unable to use the airport. So that strict design guidelines must be followed when planning runways, with particular criteria for the length, width, orientation (direction), configuration (of multiple runways), slope, and even pavement thickness of runways, as well as the immediate airfield area surrounding the runways to assure that there are no dangerous obstructions preventing the safe operation of aircrafts. Runway operations are facilitated by systems of markings, lighting systems, and associated airfield signage that identify runways and provide directional guidance for aircraft taxiing, takeoff, approach, and landing. The design and operation of runways are determined in part by the type of aircraft using the runway. (Alexander, 2004).

4.1.1 Runway orientation

Runway location and orientation are paramount to airport safety, efficiency, economics, and environmental impact. The weight and degree of concern given to each of the following factors depend, in part, on: the airport reference code; the meteorological conditions; the surrounding environment; topography; and the volume of air traffic expected at the airport(FAA, AC 150/5300-13).The appropriate direction to take off an aircraft was into which ever way the wind was blowing. Table 7 shows the orientation of the airports in Bangladesh.

Table 7: Runway Orientations of international airports in Bangladesh

Orientation	Hazrat Shahjalal International Airport, Dhaka	Shah Amanat International Airport, Chittagong	Osmani International Airport, Sylhet
Designator RWY	14	05	11
NR	32	23	29
True & MAG	144 deg. True	049 deg.True	114 deg. True
BRG	324 deg. true	329 deg. true	229 deg. true

3.2.1. Runway length and width

Aircrafts require given minimum distances to accelerate for takeoff and to decelerate after landing, runways are planned with specific lengths to accommodate aircraft operations. Characteristics that determine the required length of a runway includes most notably airport elevation above mean sea level, temperature, wind velocity, airplane operating weights, takeoff and landing flap settings, runway surface condition (dry or wet), effective runway gradient, presence of obstructions in the vicinity of the airport, and, if any, locally imposed noise abatement restrictions or other prohibitions (FAA, AC 150/5325-4B). Generally most air carrier jet aircraft require between 6,000 and 10,000 feet of runway length for takeoff at a typical airport located at sea level. Many smaller general aviation aircraft have the ability to utilize runways as short as 2,500 feet (or in some cases even shorter). As with runway length, the width of a runway is determined by the design aircraft. Specifically, the wingspan of the largest aircraft performing 500 annual itinerant operations determines the width of a runway. Runway widths at public-use airports vary from 50 to 200 feet, whereas the most common runway width planned to accommodate commercial service air carrier operations is 150 feet. The length and the width of the runways of three international airports of Bangladesh are shown in Table 8 which fulfills the standard requirements.

Table 8: Runway width and length

	Hazrat Shahjalal International Airport, Dhaka	Shah Amanat International Airport, Chittagong	Osmani International Airport, Sylhet
Length, m	3200	2940	2591
Width, m	45	45	46

4.1.2 Runway markings

There are three types of markings for runways: visual, non-precision instrument, and precision instrument. These marking types reflect the types of navigational aids associated with assisting aircraft on approach to land on the runway. The available runway markings in three international airports are shown in Table 9.

Table 9: Runway markings

	Hazrat Shahjalal International Airport, Dhaka	Shah Amanat International Airport, Chittagong	Osmani International Airport, Sylhet
Runway Markings	Marking aids at the THR, TDZ, Centre Line, Fixed Distance, and Side Stripe.	Marking aids at THR, TDZ, and center line and the taxiway marking aids at TWY holding position, TWY center line.	Runway marking aids at THR and Centre Line and the taxiway marking aids at taxiway center line.

4.1.3 Runway lighting

Runway lighting is extremely important for night time aircraft operations or in poor visibility weather conditions. Runway lighting systems are placed into three categories, approach lighting systems, visual glide slope indicators, runway end identifiers, runway edge light systems, and in-runway lighting systems (FAA, section 2). The lighting systems used in the airports are shown in Table 10.

Table 10: Runway lighting system

	Hazrat Shahjalal International Airport, Dhaka	Shah Amanat International Airport, Chittagong	Osmani International Airport, Sylhet
APCH LGT type LEN INTST	Precision approach CAT-I lighting system Simple approach lighting system and sequenced flashing lights Supplementary approach	Simple approach lighting system	Simple approach lighting system
THR LGT color WBAR	Green supplemented by green Wing bar	Green	Six Green LGT
VASIS (MEHT) PAPI	PAPI 3 deg. MEHT on slope 67ft	3 deg. PAPI	PAPI
TDZ LGT LEN	Available	Nil	Nil
RWY center line LGT	Available	nil	Nil
RWY edge LGT LEN, spacing, color INTST	60 m apart white omnidirectional with intensity 3%, 10%, 30%, 80%, 100%	Last 2000ft amber rest white omnidirectional with intensity 20%, 40%, 60%, 80%, 100%	60m apart, 79+4=84 lights, intensity 100%, 80%, 60%
RWY end LGT color WBAR	Red unidirectional Green omnidirectional	Red unidirectional Green omnidirectional	Available
SWY LGT	Available	nil	Nil

4.2 Taxiway

A taxiway's geometry and operational use play a crucial role in enhancing airfield safety and efficiency. Taxiways are identified as parallel taxiways, entrance taxiways, bypass taxiways, or exit taxiways. The widths of taxiways are planned according to the type of aircraft in use. Specifically, the wingspan of the design aircraft is used as the primary planning characteristic for taxiway widths. Taxiway widths range from 25 feet for the smallest general aviation aircraft to 100 feet for aircraft with the largest wingspans (Kazda, 2004). Taxiway widths in Bangladesh are as in Table 11.

Table 11: Taxiway widths

	Hazrat Shahjalal International Airport	Shah Amanat International Airport, Chittagong	Osmani International Airport, Sylhet
Taxiway width, m	75	115	75

4.3 Navigational Aids (NAVAIDS) Located on Airfields

Various types of navigational aids (NAVAIDS) are in use today to aid aircraft both to fly between locations and to approach an airport for landing, particularly in poor weather conditions. Some of the commonly used navigation systems are Non-directional Radio Beacons (NDB), Very-High-Frequency Omni directional Range Radio Beacons (VOR), Instrument Landing Systems (ILS), Microwave Landing Systems (MLS), and GPS Local Area Augmentation Systems (LAAS). The navigation systems used in the airports of Bangladesh are given in Table 12.

Table 12: Airport navigation systems

Airports	Navigation types used	Frequency used	
		Highest	Lowest
Hazrat Shahjalal International Airport, Dhaka	DVOR, DME (En-Route), NDB, ILS/LLZ, ILS/GP, MM, OM, OL	1161 MHz	298 kHz
Shah Amanat International Airport, Chittagong	DVOR, DME (En-Route), NDB, ILS/LLZ, ILS/GP, TDME, MM	1003 MHz	287 kHz
Osmani International Airport, Sylhet	DVOR, NDB, ILS/LLZ, ILS/GP, DME	1003MHz	372kHz

4.4 Airport Terminals and Ground Access

The airport terminal area, comprised of passenger and cargo terminal buildings, aircraft parking, loading, unloading, and service areas such as passenger service facilities, automobile parking, and public transit stations, is a vital component to the airport system. The primary goal of an airport is to provide passengers and cargo access to air transportation, and thus the terminal area achieves the goal of the airport by providing the vital link between the airside of the airport and the landside. The terminal area provides the facilities, procedures, and processes to efficiently move crew, passengers, and cargo on to, and off of, commercial and general aviation aircraft. Among the three international airports in Bangladesh the HSIA is well equipped with these facilities but the in the other two airports these facilities are not fully available.

5. AIRCRAFT CHARACTERISTICS

At present Biman Bangladesh has several modern aircrafts in the fleet such as, 2 Boeing 777-300ER, 2 Airbus A310-300, 2 Boeing 737-800, and 2McDonnell Douglas DC-10-30.Recently eight new aircrafts are going to be added in the fleet which includes two B777-300ER, four B787 Dreamliner and two B737-800. Characteristics of this aircrafts are shown in Table 13.

Table 13: Aircraft characteristics

Aircraft Type	Wing span (m)	Approach Speed (knots)	Takeoff Gross Weight (lbs.)	FAA category (Wing span)	ICAO category (Wing span)	ATC Category (Approach speed)	Wake Vortex Category (Takeoff Gross Weight)
B777-300ER	64.80	149	775,000	E	E	D	Heavy
A310-300	43.9	139	360800	D	D	D	Heavy
B737-800	34.3	142	174,200	C	C	C	Heavy
DC 10-30	50.4	149	572000	D	D	D	Heavy
B787	60.12	143	502,500	E	E	D	Heavy

6. RUNWAY PAVEMENT CHARACTERISTICS

Pavements are designed and constructed to provide durable all-weather traveling surfaces for safe and speedy movement of people and goods with an acceptable level of comfort to users. The design of airport pavements is a complex engineering problem that involves a large number of interacting variables. The two major considerations in the structural design of airport pavements are material design and thickness design.

6.1 Existing Runway Pavements of the Airports in Bangladesh

Hazrat Shahjalal International Airport (HSIA) was first built in 1979 as a rigid concrete pavement which was given an asphaltic overlay in 1994. This pavement was given a 2nd asphaltic overlay in 2012-2013. The existing pavement layer characteristics are given in Table 14.

Table 14: Existing Pavement Layer Characteristics

Pavement Layer	Thickness	E-modulus (MPa)
Asphaltic concrete	155mm	1000
Cement concrete	330mm at both ends and 305mm for middle of runway	2400
Lean concrete	152mm	1850
Subgrade soil	Unlimited	72.5

The existing runway pavements of Shah Amanat International Airport (SAIA) and Osmani International Airport (OIA) are bituminous concrete layers. The Runway Pavements are being deteriorating fast. Life Cycle of previous Asphalt Overlay works has been exhausted and Pavement surface shows sign of serious distress.

7. FAA AND ICAO REQUIREMENTS

FAA suggests that when the MTOW of listed airplanes is over 60,000 pounds (27,200 kg), the recommended runway length is determined according to individual airplanes (AC 150-5325/4B). According to Table 13 the critical aircraft for the airports of Bangladesh is B777-300ER which has span width of 64.8 meter, gross weight 775000lb and approach speed 149 knots. This belongs to the design group E according to the FAA and ICAO. These aircraft is also classified as heavy according to wake-vortex category. According to FAA the minimum width of runways should be 100ft. Again. The Boeing 777-300ER needs a runway that is 11,200ft long to takeoff at MTOW (Maximum Takeoff Weight). The 777-300 needs a runway that is 8,100ft long to land at MLW (Maximum Landing Weight).The existing runway orientation is good enough according to FAA standards. But the increasing travel demand trend demands another runway in HSIA, Dhaka.

There should provide adequate sign and markings to navigate the aircrafts on ground. FAA and ICAO suggest applying the following runway and taxiway markings. The runway markings are Runway designation markings, Runway centerline markings, Threshold markings, Aiming point markings, Touchdown zone markings, Runway edge markings and the taxiway markings are Taxiway centerline markings, taxi-holding position markings, taxiway intersection markings (Kazda, 2004).Although the airport marking system is adequate for HSIA but not adequate for the SAIA and OIA and hence should be improved. The lighting and navigation system is quite satisfactory for all the international airports. The other facilities should be improved according to standard standards.

Although there are some long haul aircrafts in the fleet but most of them are old and going to cross their service life. So that some modern aircrafts should be included in the fleet. The existing runway pavements should be evaluated for the new generation aircrafts. As the deteriorating condition of the runways continue to become harmful day by day for safe operation of Aircrafts on the Runway, construction of Asphalt Concrete overlay work on the existing Runways is necessary for SAIA and OIA.

8. CONCLUDING REMARKS

Statistics show that the popularity of air travel is increasing day by day as well as the air travel demand. On the other hand new technologies are adopted in the aviation industry so that wide bodied and long haul aircrafts are also introduced. To facilitate these factors the airports of Bangladesh should be modernized and equipped with the improved facilities. This paper will help the authority for performing the improvement works.

9. REFERRENCES

Alam, J.B. and Karim, D.M., (1998). "Air travel Demand model for domestic Air Transportation in Bangladesh." Journal of Civil Engineering, IEB, Bangladesh, Vol. CE 26, no.1.1998.

Alam, J.B., Haque, M.M., and Karim, D.M. (2001). "Potential of Privatization of Domestic Air Transportation in Bangladesh" available at: http://rahatfoundation.info/pdf/aitconference.pdf, 2001.

Alexander, T. Wells, A.T. and Young, S.B., (2004). "Airport Planning & Management", 5th Edition. New York: McGraw-Hill, 2004.

Ashford, N.J. Mumayiz, S. and Wright P.H. (2011) "Airport Engineering Planning, Design, and Development of 21st Century Airports" , Fourth Edition, United States of America,2011.

Civil Aviation Authority of Bangladesh (CAAB) (2013). "Airports in Bangladesh", available at: http://www.caab.gov.bd/adinfo/airports.html, Accessed on 14-07-2013 at 22.15.

Euromonitor International, "Travel and Tourism"(2013) available at: http://www.euromonitor.com/travel-and-tourism- in-bangladesh/report ,accessed on 20-11-2013 at 13.50.

Federal Aviation Administration (FAA) (2005). "Runway Length Requirements", AC 150/5325-4B.

Federal Aviation Administration (FAA) (1989). "Runway Location and Orientation", AC 150/5300-13.

Federal Aviation Administration (FAA) (2013). "Lighting Aid and Obstruction NOTAMs", Section 2. Available at: http://www.faa.gov/air_traffic/publications/atpubs/ntm/not0502.html, accessed on 14-11-2013 at 10.30AM.

Indexmundi (2013). "Bangladesh Air transport", Available at: http://www.indexmundi.com/facts/bangladesh/air- transport, accessed on 14-07-2013 at 23.00.

International Civil Aviation Organization (ICAO) (2002), "Traffic Statistics Database, Montreal: International Civil Aviation Organization". available at: http://www.icao.int/Newsroom/Presentation%20Slides/ICAO%20Aviation%20Data%20Presentation_NT_29_Sep2010.pdf

Kazda A. and Caves R. (2004), "Airport Design and Operation." 1st Ed. New York: Pergamon. 2004.

Robert. H.J. (1994). "Planning and Design of Airports." 4th Ed. New York: McGraw-Hill, 1994.

Routes online (2012). "Stats: Air capacity Data Bangladesh", available at: http://www.routesonline.com/news/29/breaking-news/140863/stats-air-capacity-data-bangladesh, Accessed on 11-10-2012 at 7.27.

Wikipedia (2013). "Airports in Bangladesh", available at: http://en.wikipedia.org/wiki/Template:Airports_in_Bangladesh, accessed on 15-07-2013 at 20.35.

Wikipedia (2013). "Osmani International Airport", available at: http://en.wikipedia.org/wiki/Osmani_International_Airport, accessed on 14-07-2013 at 22.35.

NOTATIONS

- HSIA- Hazrat Shahjalal International Airport, Dhaka
- SAIA- Shah Amanat International Airport, Chittagong
- OIA- Osmani International Airport, Sylhet
- FAA- Federal Aviation Administration
- ICAO- International Civil Aviation Organization
- RWY- Runway
- LGT- Lights
- VASI- Visual approach slope indicator
- THR- Threshold
- TDZ-Touchdown zone
- PAPI- Precision approach path indicator
- LEN- length
- DVOR-Doppler VHF Omnidirectional Range
- DME-Distance measuring equipment
- NDB-Non-directional beacon
- ILS/LLZ-Instrument landing system/ Localizer
- ILS/GP-Instrument landing system/ glide path
- MM- Middle Marker
- OM- Outer Marker
- OL- Outer Line
- MTOW- Maximum Takeoff Weight
- MLW- Maximum Landing Weight

ESTIMATION OF ROAD TRAFFIC ACCIDENTS COST IN KHULNA METROPOLITAN CITY

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ABSTRACT

Road traffic accidents are one of the major causes of death, injuries and disabilities in all over the world. Every year about 1.2 millions of peoples are died and nearly fifty millions are seriously injured or permanently disabled by road traffic accidents which creates an enormous burden not only on society but also economy. In this study an attempt has been made to present some of the findings of major study that undertaken to establish a methodological framework and to estimate the economic costs of road traffic accidents in Khulna Metropolitan City, Bangladesh. Using the human capital approach that focuses on the gross output of road accident victims, costs are classified into some components namely: property damage, medical cost, pain, grief and suffering cost and administration cost. The monetary value of each of these components has been estimated in this study in accordance with three types of accident severity viz. fatal, grievous and simple injury. The preliminary estimation revealed that the total annual cost of road traffic accidents and injuries in Khulna is about 0.0004 percent of total Gross Domestic Product (GDP) of Bangladesh which could be around 20 million BDT.

Keywords: *Road Traffic Accidents, Gross output method, Monetary Value, Gross Domestic Product.*

1. INTRODUCTION

1.1 General

Road traffic accidents in developing countries tend to be the major cause of fatalities and disabilities. The development and planning of transport services is a means of development. In this respect, the role of road transport is increasing than the other mode of transport (viz. rail, water and air) in Bangladesh day by day. Studies conducted by WHO and World Bank (1990) have estimated that about 500000 people lose their lives as a result of road traffic accident and over 15 million suffer injuries (Mannan and Karim, 1998). The majority of these deaths (about 70%) occur in developing countries, 65% of deaths involve pedestrians and 35% of pedestrian deaths are children. The "Study Global Burden of Disease" undertaken by the World Health Organization (WHO), Harvard University, and World Bank, showed that traffic accidents were the world's ninth biggest cause of deaths during 1990. The study forecasts that by the year 2020, road accidents would move up to third place in the table of major causes of deaths and disability (Murry and Lopez, 1994). This problem draws significant attention in Bangladesh where road accidents are extremely high and still increasing.

Moreover most people consider road traffic accidents are unavoidable in road transport systems. However, identifying and determining the significance, magnitude and nature of the problems may only change such attitudes. Road users and people in general, decision makers and politicians in particular, have to acknowledge the road hazard problems in their true proportions: as a public health problem, as an economic problem, as a social problem and as a traffic problem. In this respect determining the cost of road traffic accidents and the value of preventing them is very crucial with the following points of view. The first need for accident cost valuations, therefore, is at the level of national resource planning to ensure that road safety is given adequate priority in terms of investment in its improvement. A second need for road accident cost figures is to ensure that the best use is made of any investment and that the best (and most appropriate) safety improvements are introduced in terms of the benefits they might generate in relation to their cost.

Over the past periods, till now, traffic accident costs in Bangladesh have been estimated by the police on the loss of property damage and the general economic loss to the country has been estimated as one percentage of the GDP. However, this type of costing road traffic accident significantly underestimates the actual loss. This leads to underestimation of economic effects of road traffic accidents by decision makers and hence gives less priority to road safety issues. Consequently, estimating the costs of road accidents is important to highlight the total economic loss to the country to raise the recognition of road traffic accident problem in the country. In this study an attempt has been made to estimate the cost of road crushes and causalities and assess the annual economic loss of Khulna metropolitan city due to road accident.

2. METHODOLOGY

There are about six practical techniques or methods are available to determine the costs of road traffic accidents. However, no single costing method is believed to be an ideal method to use, and a considerable amount of data still needs to be collected regardless of the method used (Jacobs, 1995). This method basically requires the estimation of direct and indirect costs incurred to individuals and society as a whole. Generally the costs of a traffic accident are divided into two main categories: (i) the costs due to the loss of current resources, including the costs of vehicle damage, medical treatment, and administrative costs, and (ii) the costs due to the loss of future resources that the victim would have lived to earn which must be discounted back, to give present values (Jacobs1995). Usually, a significant sum is added to reflect the “pain, grief, and suffering” of the accident victim and to those who care for. These cost components can also be classified as casualty related costs (lost output, medical costs, and pain, grief, and suffering), and accident related costs (property damage and administration). The cost of an accident is therefore the sum of the casualty related costs, plus the accident related costs while the total cost of accidents in the country is the number of accidents by severity multiplied by their respective accident costs. The detail of the various cost components can be explained one by one as follows:

2.1 Medical Cost

The medical costs resulting from road accidents arise from hospital treatment (in-patient and out-patient), treatment by general practitioners and the use of ambulances. The total costs was determined by the summation of the average length of stay in hospital, the average cost per day of hospital treatment, the average number of out-patient visits, the average cost per out-patient visit, the average costs incurred by doctors and nurses, the cost incurred by ambulance service and the cost of medicine. All these factors were considered in the case of serious injuries; out-patient and practitioners treatment were ignored in the case of fatalities, and by definition in-patient costs can not arise in the case of slight injuries. All of this information was collected from published document of Ministry of Health.

2.2 Loss of output cost

In order to determine “lost output”, certain assumptions were made. In the case of fatal accidents the number of “person years lost”, was obtained by obtaining the average age of accident fatalities and subtracting from the average age at which a person ceases to work. In case of serious accidents, estimates were obtained of the average number of days that the injured person spends in hospital and then spends recovering at home from the accident. In case of a slight accident, an estimate was obtained of the (relatively small) number of days that the person was not working due to attending a doctor’s surgery, a clinic or hospital (as an out-patient) to receive treatment for their minor injury, or being at home for convalescing. Information on days lost following serious and slight road accidents were obtained from hospital records and from interviewed with the hospital workers. Loss of output due to permanent and long term injuries depends on the number of cases, the length of absence from work and the percentage disability when work is resumed. In order to estimate of the average number of days and years lost of road accident, the value of those days and years lost should be determined. This was obtained by using the published documents by the government of national wage rates, before the removal of taxes. It is important to note that it is accidents by degree of severity that are being valued but that lost output is obtained on a “a person-injured” basis. The average number of persons injured per type of accident taking place must then be obtained.

2.3 Cost of Damage to Vehicles

The information of cost of damage of vehicles was collected from the Sadharan Bima Corporation, Khulna Branch, Bangladesh. An estimate was obtained of the total number of damage-only accidents taking place. In most of the countries these was not reported to the police and accurate statistics were unavailable. Having collected information on the total cost of repair of vehicles involved in accidents, the number of vehicles involved in these accidents, the average cost of vehicular repair per accident was obtained.

2.4 Administrative and Police Costs

Police and administration costs are low compared to other cost components. These costs consist of traffic police service cost, emergency response service cost, cost of insurance and court administration. Source of the data is from the police service, courts and insurance company. According to experience of other countries, the insurance and administration account for just 2.8% of non-casualty based costs and police costs accounted for only 0.6% of all non-casualty based costs.

2.5 Pain, Grief and Suffering Costs

Although it is difficult to express pain, grief and suffering in monetary terms. The implementation of such an approach in a developing country is not easy. The ideal willingness-to-pay based costs and values should be approximated by adding an allowance for "pain, grief and suffering" to gross output figures.

3. RESULTS AND DISCUSSIONS

This study includes all the reported accidents of Khulna metropolitan city of year 2010 and 2011 from Accident Research Institute, BUET and other organization. According to the collected information a total of 38 and 28 accidents occurred in Khulna metropolitan city during the year 2010 and the year 2011, respectively. A total number of 49 vehicles were involved in road accidents during the year 2010 and 2011. The number of persons (pedestrians, passengers, drivers and other road users) involved with fatal accidents includes 132 (about 58%) and that of injury accidents includes 96 (about 42%). The cost estimation are described below:

3.1 Property Damage Cost

Detail information of property damage was collected on total 49 claims paid for different motor vehicles involved in road accidents for the year 2010 and 2011. As the table shows the collected data are summarized as the claims paid for different type of vehicles, from this data it can be produced that average claims paid for each vehicle type. Table 1 shows the collected data of claims paid for different types of vehicles in the year 2011.

As it is seen from the Table 1 that about 41% of the total vehicle cost is contributed by bus, motorcycle and heavy truck have a share of about 9% and 18% of the total claims, respectively. This indicates that any measures taken to reduce bus accidents will significantly bring a huge economic savings for the country. It is estimated that about 45% loss of the total claims paid is for fatal accidents and 40% and 15% for grievous and simple injury accidents, respectively.

Table 10: Summary of claims paid for different types of vehicles in the year 2011

Type of vehicles	Total number of accidents	Claims paid (BDT)	Average claims paid (BDT)/vehicle
Cars	2	16000	8000
Bus	13	1,43000	11000
Minibus	2	12000	6000
Microbus	1	7000	7000
Baby taxi	5	25000	5000
Motorcycle	4	32000	8000
Truck	1	7000	7000
Heavy Truck	8	64000	8000
Oil tank	2	6900	3450
Pickup	3	12000	6000
Jeep	1	4000	4000
Others	7	25000	3571
Total	49	3,53,900	-

Source: Sadharon Bima Corporation, Khulna

There is also some amount in property damage required for towing or carrying the affected vehicle to the garage from the accident spot. This amount has been added to the cost of vehicle repairs. Moreover because of the gap between companies and traffic police offices there is huge lack of information.

3.2 Lost Output Cost

This study found that the average age of fatalities and also injuries is 31 years. Using the average productive age 59 years in Bangladesh, the average lost economic years of fatalities and injuries as 28 years was deducted. Table 2 shows the years of lost output with respect to age group.

Table 11: Years of Lost Output with respect to Age Group

Age Group	Average Fatal Age	Years of Lost Output
< 7 Years	3.5	55.5
7-13 Years	10.0	49.0
14-17 Years	15.5	43.5
18-30 Years	24.0	35.0
31-50 Years	40.5	18.5
> 51 Years	55.0	4.0

The monetary values of lost years were obtained by using the annual per-capita income of individual in the nation. Table 3 shows the national GDP of Bangladesh and population of Khulna. It is seen that the GDP of Bangladesh for the fiscal year 2010-2011 is 4093.72 Billion BDT (Bangladesh Bank, 2011) and the projected population (based on the 2001 population census) for this period is 1,046,341.

Table 3: National GDP and Population of Khulna

Fiscal Year	Population (million)	GDP at Current Market Price (Billion BDT)
2001-02	1.172	2501.81
2002-03	1.165	2655.12
2003-04	1.151	2848.98
2004-05	1.144	3032.07
2005-06	1.137	3217..86
2007-08	1.130	3406.52
2008-09	1.123	3608.45
2009-10	1.116	3850.5
2010-11	1.046	4093.78

Source: Bangladesh Bureau of Statistics and Bangladesh Bank

Lost labor output of fatalities and injuries was computed by using the cumulative present values of the estimated wages of the tabulated lost years of the victims. Short term lost wages refers to economic output lost due to non-productivity as a result of an injury or spent time recovering from an injury. Table 4 shows the lost output based on casualty class.

Table 4: Lost Output based on Casualty Class

Severity Level	BDT
Fatal Accident	3,698,735
Grievous Accident	2,090,026
Simple Accident	6,536
Total	5,795,297

Average monthly income of pedestrian was assumed about BDT 3700. For caring of victims for fatality, grievous and simple injuries was 21, 51, and 17days, respectively and the careers daily income loss was BDT 472 for fatality and grievous injuries and BDT 118 for simple injuries (Roads and Highways Department, Bangladesh, 2004).

Lost Output = (Passenger's + Pedestrian's + Career Lost Output)

3.3 Medical Costs

Medical costs only usually constitute a small proportion of the total costs of crashes. However, the medical cost is the first and most tangible economic burden experienced by the family. Medical costs were derived from actual payments made by patients in Khulna Medical College Hospital (KMCH). Table 5 shows the medical costs. It is seen that the average rescue service consumes BDT1800; average cost per day cost is BDT1096;

which includes all tests and food, for each fatality average staying at hospitals is 21days, for grievous injury hospital stay is maximum 90 days and for outpatient treatment it is about 17 days. On an average the cost of radiology per patient (X-Ray & CT-Scan) is calculated to be BDT150.

Table 5: Medical Costs

Item	2010-2011		
	Fatal	**Grievous**	**Slight**
Patient treatment cost per day (BDT)	500	500	-
Patient stay time in hospital (Day)	21	90	-
Overhead cost per patient (BDT)	355	1096	-
Operation cost per patient (BDT)	500	3000	-
Radiology cost per patient (BDT)	500	500	-
Total medical cost per person (BDT)	25132	40352	-
Out-patient treatment cost per person	-	-	3600
No. casualties	58	18	11
Total medical cost (BDT)	1457656	726336	39600

Source: Khulna Medical College Hospital (KMCH)

3.4 Police and Administrative Costs

It is usual in previous international studies that police and administration costs are low compared to other cost components (Haque *et al.,* 2008). The reason being is these costs are not direct costs that can be associated to accidents. Police investigation, insurance administration, legal cost and accident spot cleansing are included to estimate the administrative costs. Table 6 shows the administrative cost per vehicle.

Table 6: Administration cost per Vehicle

Type of Accident	Average event related Administration Cost (BDT)	Others Administration Cost per Vehicle (BDT)	Adjusted Total Administration Cost per Accident (BDT)
Fatal RTA	2,750	8,641	11391
Grievous RTA	2,750	7,583	10333
Simple RTA	1,600	6,599	8,199

3.5 Pain, Grief and Suffering Costs

Although, it is difficult to express pain, grief and suffering in monetary terms, but estimation of the costs of accidents which directly or indirectly fall upon individuals suffering bereavement is necessary. In the absence of more detailed research targeted at societies and economies in low income countries 60% of total lost output where total lost output is summation of Passenger's and Pedestrian's Lost Output has been taken (Haque *et al.* , 2008). The pain, grief and sufferings cost is calculated by the following formula. Table 7 shows the pain, grief and sufferings costs.

Pain, Grief and Sufferings =Lost Output= (Passenger's + Pedestrian's Lost Output + Career lost Output) * 0.60.

Table 7: Pain, Grief and Sufferings costs according to casualty class

Casualty class	BDT
Fatality	2,219,241
Grievous Injury	1,254,016
Simple Injury	3,922

3.6 Adjusting Resource costs for Under-Reporting

As stated earlier it is expected that significant under-reporting of injuries exists in developing countries like Bangladesh. A study conducted by Ross (2001)have raised the total cost of fatal accident by 25% ,and have increased serious and slight injuries by 100% meaning the values reported by police statistics will be doubled when under-reporting is considered whereas the study proposed a ratio of 5:1 as a ratio of damage only to injury accidents considering the effect of under-reporting In the absence of under reporting information the total cost estimated using the police statistics should be adjusted by 1.25 times for fatal, 2 times for grievous and simple injuries.

3.7 Summary of Total Accident Cost

The results of all of the computations for accident cost components are summarized in Table 8. It is seen Table 8 that among the total cost for different type of accident fatal accident take the major part (about 52%) and the grievous and simple accident is 47% and 1% of total accident cost respectively.

Table 8: Distribution of cost components based on severity level

Cost Component	Accident Type			Total Cost (BDT)
	Fatal RTA	Grievous RTA	Simple RTA	
Medical Cost	1,457,656	726,336	39,600	2,223,592
Vehicle Damage Cost	159,255	123,865	53,085	336,205
Lost Output Cost	3,698,735	2,090,026	6,536	5,795,297
Pain, Grief &Suffering	2,219,241	1,254,016	3,922	3,477,179
Administration Cost	11,391	10,333	8,199	29,923
Total	7,546,278	4,204,576	111,342	11,862,196
Adjustment for underreporting	(1.25 times)	(2 times)	(2 times)	18,064,683
Total Cost (BDT)	9,432,847.5	8,409,152	222,684	
Total cost (%)	52.22	46.55	1.23	100

Table 9 shows the comparison of total accident cost with GDP of Bangladesh at current market price. It is found that accident cost in Khulna metropolitan city is about 0.0004% of total GDP. This amount is around 20 million BDT, which is very high in context of Khulna metropolitan city.

Table 9: Total Road Traffic Accident Cost in Khulna metropolitan city as proportion of GDP

Item	Million BDT
Estimated Total Accident Cost	18.064
GDP at current market price	4,093,780
Accident cost in % of GDP	0.0004

4. CONCLUSIONS

This study shows that according to casualty class total cost share by fatalities, grievous injuries and simple injuries are 43.43%, 56.32% and 0.25%, respectively. A fatal accident consumes about 68 percent of costs and 24 percent in grievous accidents and less percentage is for simple and property damage only. The national economic losses resulting from road accidents in Bangladesh are considerably high, and so on in Khulna metropolitan city even if the conservative human capital method is employed in estimating. Based on this estimated annual road accident cost, it can be concluded that road accidents do not cause only losses in lives of productive members of the population and a substantial number of disabilities and injuries but also generate a great loss to the country's economy. It is timely to urge all agencies concerned to put forward more efforts, as well as sufficient manpower and other resources, to effectively address the road traffic accident problems. Travel time delay from accidents and cost for fuel loss due to congestion created due to accidents can be considered for future research.

5. REFERENCES

Asian Development Bank (2003). "Road Safety Guidelines for the Asian and Pacific Region: Road Accident Costing", Asian Development Bank.

Haque, M.M. (2006). "An Estimate of Road Accident Costs in Singapore", Proc. Of the International Conference on Road Safety in Developing Countries, Dhaka, Bangladesh, pp. 28-35.

Islam, S. (1996), "Traffic Accidents in Khulna City", Master's Thesis, Department of Civil Engineering, Bangladesh University of Engineering & Technology, Dhaka.

Mannan, M.S. and Karim, M. (1998). "Road accidents in metropolitan Dhaka, Bangladesh". IATSS Research, Japan, Vol. 23, No. 2, pp. 90-98.

Murray, C.J.L. and Lopez, A.D. (1994). Global and regional cause of death patterns in 1990", Bulletin of the World Health Organization, 72(3), pp. 447-480.

Ministry of Communication (2005). "RHD Road User Cost Annual Report for 2004-2005", Economic Circle, Roads and Highways Department, Bangladesh.

Paul, S., Haque, M. M., and Mahmud, S.M. (2008).“ The Cost of Road Traffic Accidents in Bangladesh”, Publication in 10th Pacific Regional Science Conference Organization (PRSCO), 15-17 May 2008, Dhaka, Bangladesh, pp. 88.

Ross, S. (2001). “Study on Sectoral Road Safety Program in Ethiopia. Volume II, Transport Research Laboratory, Addis Ababa.

TRANSPORTATION SCENARIO OF DHAKA CITY: CONGESTION AND AIR POLLUTION

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ABSTRACT

One of the most challenging and complicated issues in city management in the present decade for Bangladesh is the traffic problem. Traffic congestion has now become a very serious problem particularly in metropolitan Dhaka, the capital of Bangladesh which is the largest and most industrialized city in the country with some 135 million people (STP 2005). Because of the rapid socio-economic changes and increase in population, the city expanded dynamically without any planning and control. Such rapid and uncontrolled developments have created an unacceptable level of disparity in the transportation demand and supply scenario, which resulted in traffic congestion and environmental degradation through air pollution. The issue of transportation and the environment is paradoxical in nature since transportation conveys substantial socioeconomic benefits, but at the same time transportation is impacting environmental systems. From one side, transportation activities support increasing mobility demands for passengers and freight, while on the other, transport activities are associated with growing levels of ***environmental externalities****. This has reached a point where transportation is a dominant source of emission of most pollutants and their multiple impacts on the environment. The aim of this study is to determine the current condition of transportation system in Dhaka city, and will examine the effects that traffic congestion have on the city, to analyze the causes of congestion, to assess the impact on environment due to congestion and finally tried to identify the solution of the traffic congestion.*

Keywords: *Traffic congestion, Passenger , Environmental externality , Emission, Environmental degradation.*

1. INTRODUCTION

Dhaka, the capital city of Bangladesh, is the largest and most industrialized city in the country with some 135 million people (STP 2005). It is one of the most densely populated cities of the world. The enumerated population of Dhaka is 1,20,43,977 and the growth rate of population is 3.48 (Community report of Dhaka Zila, june 2012). The demographic trends of the last decade have resulted in rapid population growth and are expected to continue in the coming decades. The impact of such rapid growth has major consequences on the ability of the transport sector to provide mobility for all people as they seek to take advantage of employment, education, health and social opportunities. Albeit Dhaka is one of the most populous cities, the transportation sector has not been able to emerge out of unplanned framework. As result transportation problem has become the number one problem of Dhaka city. The deteriorating traffic conditions are causing increasing delays, unexpected level of disparity in transportation supply and demand scenario and thus seriously compromising the ability of the transport sector to serve and sustain economic growth and quality of life.

Moreover, the rapid rise in population along with the increase in vehicle ownership has been contributing to the deterioration on the environment mainly in the form of traffic emission. As result the issue of transportation and environment has become paradoxical since transportation activities support increasing mobility demands for

passengers and freight, while on the other side, these activities are resulting in the growing levels of environmental externalities. As a result to analyze the traffic condition and provide a framework for the structured and environmentally friendly traffic movement is a inevitable step for the policy maker.
This study provide an insight about the traffic condition of Dhaka city by analyzing the motorization level and level of pollutants emission from the vehicles. It also provide a vivid description about the causes,consequences, and remedy of traffic condition of this city. The insight gained from this study will definitely provide the poilicy makers.

2. METHODOLOGY

2.1 Study area and Data

To provide an insight about the traffic condition yearly number of registered motor vehicles in Dhaka city (upto 2012) has been obtained from the Bangladesh Road Transport Authority (BRTA). Besides, the previous publications and studies have also supplemented the data and insight regarding the traffic condition.

3. RESULTS AND DISCUSSIONS

3.1 Transport System and Traffic of Dhaka

Dhaka city mainly depends on road-based transportation system but the amount of road network is far apart from the minimum requirements. The city's road network is about 1844 km in municipal area and 220 km in greater Dhaka (Rahman, 2001). It has 62 km functional primary and 108 km secondary and 221 km connector road serve the city transport service. Most of the intersections are at grade intersections. There is a flyover at Mohakhali and an interchange at Khilgaon and Kuril. Another interchange is under construction at Gulistan.

Household survey of STP (2005) reveals the primary travel mode of all trips which is shown in Table 2. This reveals that 51% use the non-motorized (walking, rickshaw) travel mode and other 49 % uses motorized (transit, non transit) mode. Again 18% uses motorized non transit (car, van, pickup, auto-rickshaw, taxi, motorcycle) mode and 31% uses the motorized transit mode. Though, a very small percentage of the trips are being served by the individualized motorized transport modes, they occupy a significantly large portion of the roadway of Dhaka city

Table 1: Primary travel mode for all trips

Walk	22%
Rickshaw	29%
Transit	31%
Motorized (Non-Transit)	18%
Total	**100**

Albeit 18% of the total trips use the BRTA report suggest the rapid growth of the motorized non-transit vehicles. The number of registered private car, microbus, jeep is 257458, number of buses are 19502, and human haulers are 3432 up to 2012 (BRTA report).
Figure 1 shows the motorization level of the motorized non-transit vehicle which reveals the rapid growth of the private passenger car. Whereas motorization level of the transit vehicles is low (Figure 2) compared to the motorized non transit vehicles. But these individualized vehicles carry fewer passengers than the bus and these results in increases number of vehicles on the road which contribute to the congestion and more emission of the toxic gases.

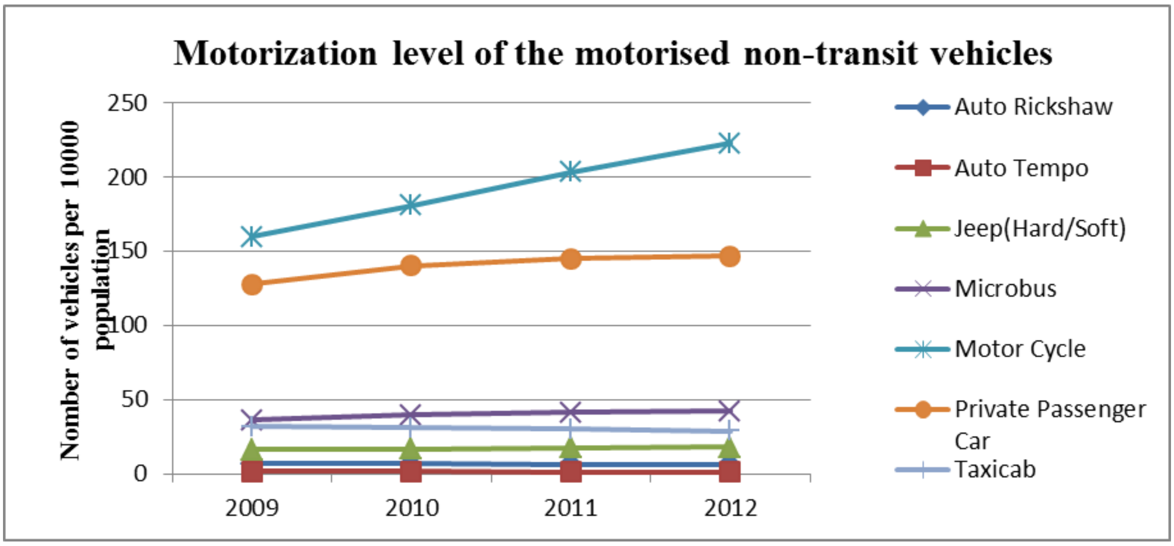

Figure 1: Motorization level of the motorized non-transit vehicles

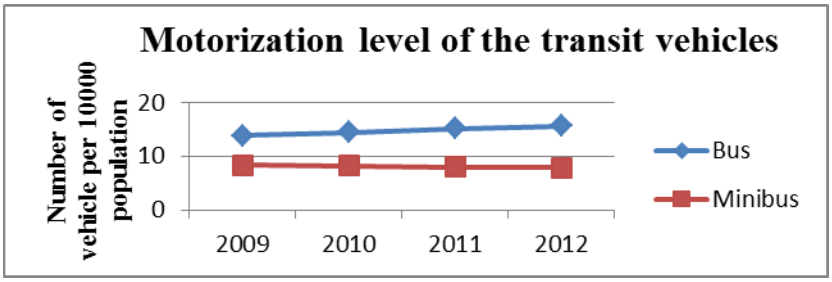

Figure 2: Motorization level of the transit vehicles

Moreover road infrastructure of the city has been emerged in an unplanned structure. Operational weakness can be identified in the route planning and infrastructure development. It is said to be the only city of its size without a well-organized, properly scheduled bus system or any other mass transport system. The transport system in Dhaka is characterized by different types of modes, with both motorized transports (MT) & non-motorized transport (NMT) using the same carriageway. As a result heterogeneous traffic condition makes the transportation system of the city more unique and complex.

In case of rail transport, ten stations are located within the city from Tongi to Kamlapur in the north region and from Narayangonj to Kamlapur in the south region. The rails in the north region carry small amount of local traffic and that of the south region are grossly underutilized (Hoque and Choudhury, 2009). Moreover, the at grade level crossing and the side friction along the rail route significantly reduce the operating speed of the vehicles.

Waterways also contribute to the peripheral movement in certain part of the city but it role in the urban transport is very insignificant.

3.2 Road Transport Management of Dhaka City

Dhaka has a multi modal transport infrastructure that caters inter-city passenger and freight traffic need. The agencies which are involved on the operating and maintaining the transport infrastructure need and services of the city are Dhaka City Corporation (DCC), Bangladesh Road Transport Corporation (BRTC), Bangladesh Road Transport Authority (BRTA), Dhaka Transport Coordination Board (DTCB), and Traffic Police.

It is evident that the transport infrastructure and services are mainly provided by the government agencies. Two major factors mainly contribute to this situation:

- Relatively high investment requirements
- High risk involvement in the provision and management of infrastructure.

As result private sectors are discouraged to from entering in this area. But involvement of the private sector in the operation and maintenance system of the transportation has been proved to increase the efficiency of the work. In such situation, government exercises control through regulatory mechanism of vehicle registration, fitness check, route permits, and price fixation in order to prevent the exploitation of the private organization.

3.3 Traffic Induced Air Pollution in Dhaka City

Dhaka grew from a provincial capital to a national capitol in an unplanned way. Moreover the huge pressure of the pressure of the huge population results in problems in different sectors. But the most significant problem is the transportation problem that causes the people of the city to suffer a lot in terms of time hammering, loss of money, inefficiency in their daily works. As a result the common terms that can be used to describe the present condition of Dhaka City are:congestion, delays, pollution, etc.

Congestion has become an issue of great concern for the inhabitants of the city resulting in commuter's frustration, longer travel times, lost productivity, increased accidents, more fuel consumption, and deterioration in air quality. A recent study by Roads and Highways Department (RHD) has estimated that, the traffic congestion in Dhaka causes a loss of Taka 19,555cr a year (The Daily Star, 2010). Several attempts have been taken to reduce the problem but as most of them were implemented without any priory research most of them has resulted in failure to solve the congestion and congestion related problems. Md. Asadullah Khan has described the scenario of traffic jam in Dhaka city in Daily Star with the title "When shall we get rid of Dhaka city traffic jam?" published on October 20, 2007 (Khan, 2007). According to him with a huge fleet of cars, buses and all other types of vehicles gridlocked near a rail gate or road intersection sometimes even for 30 minutes at a stretch. City's traffic congestion problem has has become so alarming that people are afraid to get out of their houses because the journey from home to office or business centre takes away the vital hours that he could devote to his work.

The growing number of vehicles and increasing traffic congestion contribute to the deterioration of the environment mainly to the air quality. Vehicle emissions are increasingly being recognized as the dominant cause of air pollution and health problems in Dhaka city (Bhuiyan, 2001). According to Mr. Paul Martin, a bank environmental specialist in Dhaka 2001 air pollution kills nearly 15,000 Bangladeshis each year. This is released in the World Bank report saying Bangladesh could save between $200 million and $800 million per year. These amounts translate to about 0.7% to 3.0% of the gross national product if air pollution is reduced in just four major cities of Bangladesh. Jaigirdar (1998) conducted a detailed study in regard of vehicle induced pollution of Dhaka city. According to Jaigirdar (1998), the maximum instantaneous concentrations of SO_2, NO_2, and CO are 0.7 ppm, 0.3 ppm, and 93 ppm respectively. Instantaneous concentration of SO_2 and NO_2 are high at two intersections named Gulistan and Mohakhali where diesel fuelled vehicles like busses, trucks operation are high. The concentration of CO is high where car, microbus, movements are high. Most of the road intersections are highly polluted by SO_2 and NO_2. Albeit the concentration of CO is seemed to be moderate as compared with SO_2, the concentration of CO in most of the intersections is harmful for heart patients (Stewart, 1975). Figure 4, 5, and 6 shows the SO_2, NO_2, CO emission by mode.

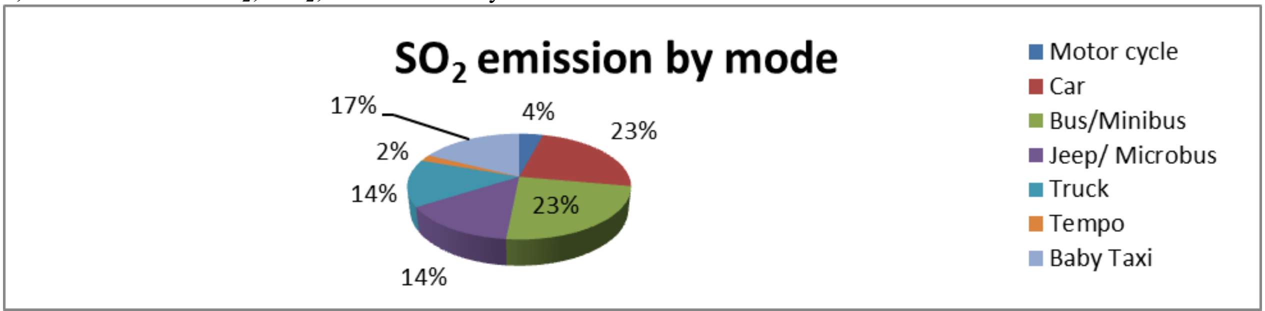

Figure 3: SO_2 emission by mode

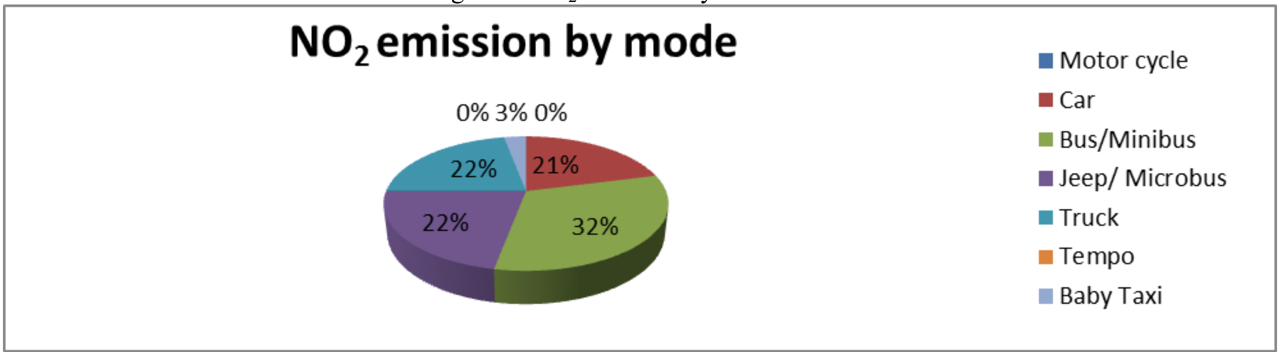

Figure 4: NO_2 emission by mode

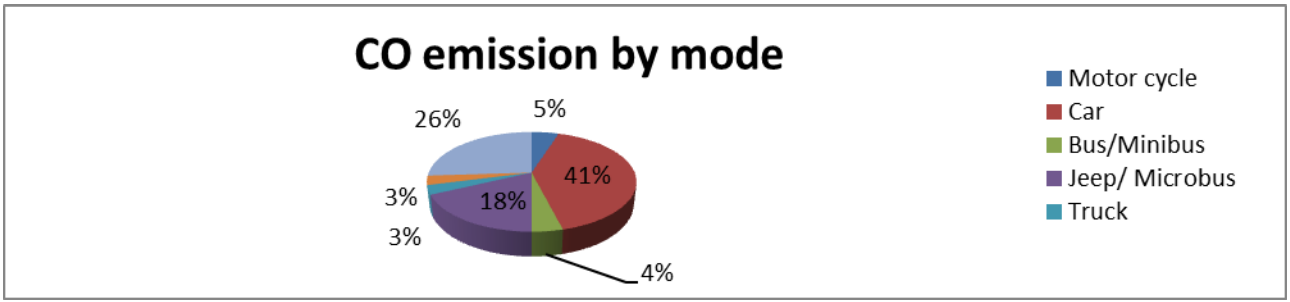

Figure 5: CO emission by mode

Source: *(Jaigirdar, 1998)*

Noise pollution is another problem which is the consequences of vehicles horns that are abused by drivers, horn used to get right of way. Strength of horn decides the power of vehicles. Existence of non motorized vehicle on the same track encourage the use of horn, the non-motorized vehicles have no side/rear view mirror compel more fast moving follower to use horn. Again most drivers like horn signal than light indicator signal for lane changing. As a result severe noise pollution occurs that causes the various health problems.The survey regarding noise pollution was performed by Geography & Environment department of Jahangirnagar University. 100 people were interviewed among different professionals like doctor, traffic police, driver, teacher, student, businessman and service holders. It was found that all of them were invaded with diseases due to excessive exposure of noise pollution.

Accidents are another consequence of rapid urbanization and increase of vehicles. Each year, there is at least 1600 police reported fatalities and 3000 injuries on Bangladesh roads. The safety problem is very severe by international standards with some 45 fatalities per 10000 motor vehicles in Bangladesh compared to 2.0 in the USA, and 1.4 in UK (Hoque et al.,1997).

3.4 Causes of the Transportation Problems

Transport has an impact in shaping the urban areas and dwellers' lifestyles (Karim, 1998). An efficient and effective urban transportation system is a means to both promoting urban development and providing adequate access and mobility to the urban dwellers. Cities of the developing countries are the major engines for economic growth, and as a result, more attention is being focused on urban development as an important part of the national growth process (Kwakye et al., 1997). Transport can release working capital from one area, which can be used more productively as fixed capital elsewhere (Button, 1993). As result transportation sector has always been the top concern for the government. Many studies, plans and projects have been undertaken costing crores of taka to overcome the problems but most of them have not proved to be effective as the most of them were untaken without any priory study or realizing the root of the problem. In the following section the causes of the transportation problem are discussed.

As discussed earlier congestion is the root of all transport related problem of the city. The causes of traffic congestion in Dhaka city can be divided into three broad categories (Habib, 2002). These are:

- Site-specific causes
- Transportation system capacity related causes
- Planning or policy related causes.

The site-specific causes are mainly traffic management related problems. The important maneuverings like left turning, through movement along the intersections etc. are often seriously obstructed for the poor driver's knowledge and in maximum cases absence of traffic management system such channelization or any enforcement. DMTP, a wing of Dhaka Metropolitan Police, mainly deals with the enforcement, operation, management and occasionally made short term policy planning like banning certain vehicle class in certain corridors, banning certain turning movements at locations, turning certain roads into one-way operation. Unfortunately, none of the traffic police personnel has either any engineering background in general or any traffic engineering background in particular. Therefore, apart from the enforcement job which should have been their sole responsibility, DMTP is unjustifiably overburdened with rest of the responsibilities. Though traffic signals have been installed at many intersections most them have been found to futile as Dhaka's road network possess inherent weakness in particular relation to road quantity, road orientation or layout, functionality and operational and management aspects. The city has been developed in so unplanned way that there is no regular pattern for intersection interval. As a result optimized traffic signaling becomes very difficult.

In addition uncontrolled parking of both motorized and non-motorized vehicles mainly near intersection in order to grab passenger easily, make maneuvers more hazardous. Buses and minibus also stop near the intersection in order to board and inboard the passenger. In many locations of the road network, the effective roadway spaces are reduced by roadside activities, presence of dustbins, hawkers etc. Such reduction in effective roadway spaces in links and intersections creates bottlenecks to traffic flow and causes congestion. Besides, poor road surface, Occupied or partially damaged footpath, roadside construction material on the road, etc. causes the pedestrian to leave the footpath and walk on the street. It not only causes safety issues for the road users but also contributes to the congestion. Besides poor drainage causes water logging during the rainy periods. It also contributes to the roadway congestion. Figure 7 & shows the showing main reasons for disruption of normal traffic operation on Dhaka city roads.

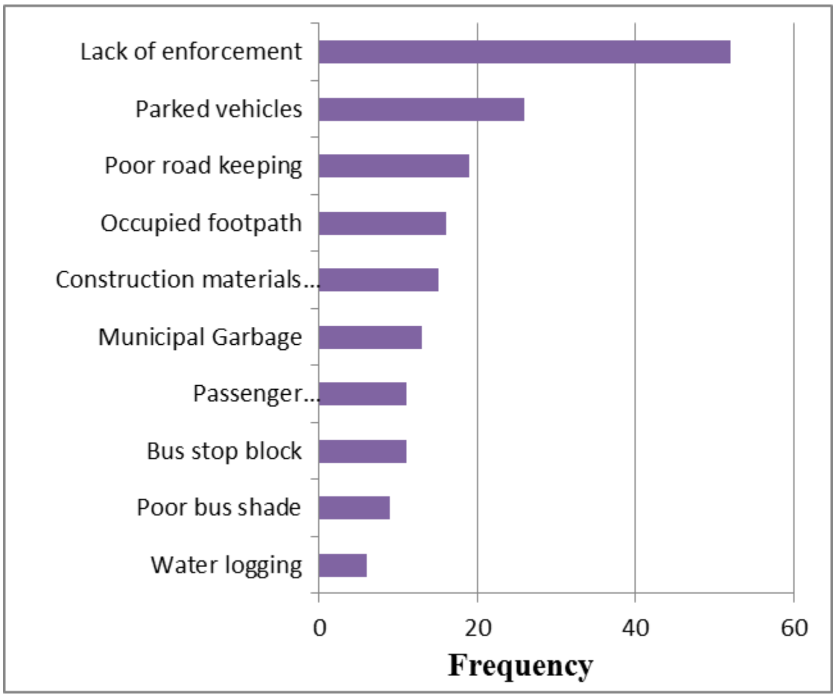

Figure 6: Main reasons for disruption of normal traffic operation on Dhaka city roads
Source: *Hossain (2004) Journal of Civil Engineering (IEB), 32 (1) (2004) 47-58*

As mentioned earlier increase of physical capacity of the city has become difficult by keeping pace with the increasing population. The lag between increasing travel demand and system capacity is resulting traffic congestion. Another important factor is the uncontrolled increase in rickshaw traffic. Rickshaw provides door to door transport facility with more comfort and safety to the users. Level of service of the present bus system is inadequate to meet the demands of public. Long waiting time, delay on regular schedule, overloading, discomfort, and long walking distance from the residence and work place to bus stops are some of the obvious problems facing the users in their daily life. In Dhaka average walking distance to bus stop (unofficial bus stop) is about 644 m and average waiting time for buses is over 15 minutes (Firdus, 1984). Moreover though, the total length of the route is 200 km, only 107 km of the available road are of less than 24 km width and there has been a lack of sufficient connector roads in the east-west direction. Also, several operational weaknesses can be identified in terms of route planning, for example, deficient number of routes, competition of numerous small operators in the same route, absence of fixed schedule and passenger information system, etc. Moreover, no advantage is practiced by these buses in terms of separate right of way or signal priority. Also, the bus passengers are vulnerable to accidents, since the bus drivers often carry false driving license leading to increased congestion and indiscipline in the road. In addition, Poor terminal facilities and narrow lanes are the main constrains to access for the cheapest modes in the depressed areas. Around 55% of the total metropolitan area is unserved by buses (Firdus, 1984). As a result the popularity of rickshaw increases with it number. Now as there is no segregation of motorized and non-motorized traffic in the network, the increasing rickshaws are occupying the maximum roadway space and compelling the other modes to move slowly and creating congestion.

Recently, step such banning rickshaw on some routes has been taken by the government. But the policy mainly benefited only small minority of Dhaka City who own a car as no priority has been provided to the public transport to increase its level of service and accessibility.

The overall transportation system of Dhaka city has not been developed in a planned way rather it has been developed in dynamic response to increasing travel demand. Even there is no definite policy to a sustainable transportation system. Uncontrolled land-use together with increasing migration of rural people towards Dhaka is increasing pressure on transportation system and creating traffic congestion and other related problems. Moreover unplanned construction of the flyover and interchange are just worsening the condition.

The pressing demands for motorized form of personal mobility are generating pressures on road network and resulting in congestion, which threatens the sustainability of the environment. Mainly motorized vehicles are the foremost reason behind air pollution but Large number of rickshaws and other vehicles in the city streets frequently causing severe traffic jam consequently cause the city dwellers to be exposed to highly polluted air for a long period and immediate surroundings of vehicle emission. On the other hand noise on roads is caused by engine of the vehicles, its exhaust, horn, brakes, friction between tires and road surface. In the city, the main sources of traffic noise are the motors and exhaust systems of autos, smaller trucks, buses, and motorcycles. This type of noise can be augmented by narrow streets and tall buildings, which produce a "canyon" in which traffic noise reverberates (Environmental Pollution Report, 1998).

Typically, the principle contributor factors of accidents in Dhaka are (Binnie Partners, 1994)

- Mix of traffic with variety of vehicle characteristics and speeds.
- Failure to obey mandatory traffic regulation illegal and inconsiderate driving practices
- Pedestrian and vehicle conflicts
- Failure to provide and maintain road signs and marking
- Failure to enforce traffic law
- Lack of education of road users
- Poor detailed design of junction and road section

Haworth (1995) identified a range of road user behavior problem that contribute to accident risk and poor traffic flow in Dhaka. The prevalent behavior problems include:

- Failure to give way
- Lack of lane discipline
- Counter-clockwise travel at roundabouts
- Non-wearing of motorcycle helmets
- Failure to slow down when approaching an intersection.

Furthermore, traffic accident rates in a congested flow, especially rear-end collisions are much higher than no-congested flows. Traffic congestion therefore, affects not only the productivity of highway operation but also safety (Iwasaki, 1992).

Actually, the absence of institutional entities and specialized transport civil engineers in the field of modern transport engineering with respect to immediate and short term transport management and long term strategic transport planning has been claimed to be one of the main causes of the persistence of transport and traffic problems in many cities of the developing world. (Huzayyin and Osman 2001)**.**

3.5 Solution of Traffic congestion and Air pollution

3.5.1 3.5.1 Solution of Traffic Congestion

3.5.1.1 Clarity in BRTA

BRTA should maintain transparency of their activity. Issuing driving license and transport fitness license should be monitored properly and proper steps should be taken in case of defaulters. Digital driving license should be introduced to control the fake driving license holding drivers.

3.5.1.2 Strict lane management

Different lanes for different types of vehicles should be marked on the roads and law i.e. financial penalty should be imposed to make the drivers maintain the lane.

3.5.1.3 Restricting routes for Rickshaw

Rickshaw should not be allowed in all the routes of the city. BRTA should take some responsibility to control the increasing number of rickshaws by imposing registration fee and legal documentation.

3.5.1.4 Financial penalty to the traffic law breakers

Government can take such strict step like imposing financial penalty on the law disobeying drivers. This trend is available in Dhaka. According to Remi, at el., (2009) policies should be made to dissuade the drivers from certain congestion-causing habit such as wrong overtaking, one way driving, disobey of traffic signals Mobile court should be introduced to fine the truck drivers for disobeying traffic law and driving unfit truck. This kind of implication of law can mitigate the traffic jam in short run, but in long run all the people should be involved to create awareness and responsible to the society. Otherwise traffic jam solution is impossible.

3.5.1.5 Elevated Express way

With over a year-long effort, the Bangladesh Bridge Authority (BBA) is yet to select the bidder to construct the DEE over the city. Initially it had been planned to give construction work without feasibility study. But the government has to scrap its decision in the face of various queries by the participating bidders. The pre-qualification bidding of the US dollar 2.0 billion project was opened on November 19, 2009 and its awarding may take one month's more time to complete. According to communication ministry the work will start in January after completing the signing of concessional agreements by this month. Till today, modalities of the agreement are yet to be finalized and the experts have still doubt about the completion of the first ever elevated expressway project before the end of the present government's tenure in 2013 as claimed by the minister. The planning should be implemented now so that Dhaka city can enjoy the elevated express way in coming decade.

3.5.1.6 Supply and demand

Congestion can be reduced by either increasing road capacity (supply) or by reducing traffic (demand) (Remi, at el. 2009). Hermann, (2006) revealed that road capacity can be increased in a number of ways such as adding more capacity over the whole of a route or at bottlenecks, creating new routes, and improvements for traffic management. Reduction of demand can include, parking restriction, park and ride, congestion pricing, road space rationing, incentives to use public transport and introduction of e-education, e-shopping and home-based working options will reduce the number of people traveling.

3.5.1.7 Bus route franchising

Bus route franchising (BRT) is a concept applied in Dhaka, The country's first-ever bus route franchise (BRF) with digital ticketing system started its journey on Uttara-Azimpur route in the city on 14th April, 2011. This process is going to reduce the hassle of the passengers. This kind of services should be increased.

3.5.1.8 Building bus stoppages

In Chittagong there are buses stoppages only where busses and other public transportation can stop and pick or drop the passengers, but these bus stoppages exist only in written form or legally. Rarely any public transportation will be found that stop according to the stoppage. Bus stoppage should be well developed in the city and the law enforcing authority should be aware of confirming the implication of law.

3.5.1.9 Increasing and developing the manpower (Traffic police)

As the city is running with inadequate amount of traffic police than required, so it is almost must for the authority i.e. CMP to increase the number of traffic police. This step will create some scope for employment also. Only recruitment is not enough, they should be trained up for the betterment of the traffic management.

3.5.1.10 Road widening

Road widening is often advocated as ways to reduce traffic congestion. Roads of the city are narrow in different places, there are several reasons like Hawkers on the footpath and some portion of the road (scenario is regular in New Market area) and illegal possession on the road or illegal structures. This kind of unlawful activity has to be prevented by imposing proper law and city development plan.

3.5.1.11 Imposing tax on car and other private transportation

Government is already charging a high amount of tax on imported cars and other private transports. Tax amount should be increased in order to reduce the amount of private transportation hence traffic congestion. But this formula is applicable after making the public transportation available, comfortable and convenient as well.

3.5.2 Solution of Air pollution

3.5.2 .1 *Cities Try to Tackle Traffic Pollution*

The solutions to reducing or eliminating traffic as a source of pollution aren't really that innovative. Cities that invest heavily in public transit can see dividends in the form of fewer cars on the road. Many people are reluctant to give up the freedom of their own vehicles, but rising gas prices often help to encourage good numbers of drivers to make the switch to the subway, light-rail or the bus. Employers can help take cars off of the road by offering incentives to employees who take public transit or carpool as well.

Still, even with effective public transit, air-quality problems persist in many areas. Cities like London have begun charging tolls to drivers who access high-congestion areas of the city during peak traffic times. Car-share companies have also started to see success in dense urban areas, spreading the cost of car ownership and — hopefully — emissions across a wider number of people, reducing trips, mileage and pollution.

3.5.2 .2 *Advancing Technology Helps Reduce Emissions*

We've already seen a large reduction in the rate of emissions generated by a single vehicle, thats largely to improved car engine designs, catalytic-converters and improved fuel chemistry. On the horizon, we can look forward to the increasing popularity of hybrid and electric technology for personal vehicles.

The challenges of these types of technology is that they are far from perfect and may just shift the pollution to a separate place in the energy chain. Electric cars, for example, simply get their energy from the electrical grid, which is generated largely by coal-burning power plants, generating its own pollutants in mining and eventual burning of coal. If cleaner sources of electricity can be brought effectively to market, such as solar power, we may be able to avoid millions of tons of emissions.

All of these solutions can help mitigate the impact of traffic on our air-quality, though they'll undoubtedly need to be combined with other strategies for cleaning up our air. Public transit and smart-driving can do a lot to reduce emissions and clear the air. Likewise, seeing smarter construction projects, ones that improve the efficiency of our highway systems by reducing stop-and-go traffic, will go a long way to get cars moving without simply inviting even more cars onto the road. In the end, contributions from conscientious drivers, innovative technology and wise city-planning will a sound environment.

4. CONCLUSIONS

Cities are the powerhouses of economic growth for any country. According to Bartone et. al. (1994), around eighty percent of GDP growth in developing countries is expected to come from cities. In order to accelerate the development of the country a sound environment and well planned Dhaka city in the transportation sector is the foremost need.

In Dhaka, different infrastructural and managerial projects are granted for reducing traffic jam and air pollution. Traffic congestion and air pollution constraints can be ameliorated by embarking on various strategies such as road capacity expansion, improved road infrastructures, restricting routes for Rickshaw, financial penalty to the traffic law breakers, building bus stoppages, application of Fly over, vehicles older than 20 years could be taken off from the city streets and faulty vehicles should be eradicated especially those that run on diesel. Most importantly, proper traffic management system along with appropriate implementation of traffic rules is necessary to mitigate the problems of traffic congestion.

5. REFERENCES:

A survey of Geography & Environment department, Jahangirnagar University, 2001-02.

Bangladesh Road Transport Authority.Registered vehicles in Dhaka city. www.brta.gov.bd.

Bari and Hasan, 2001 Bari, M. F. and Hasan, M., 2001, "Effect of Urbanization on Storm Runoff Characteristics of Dhaka City", Tsinghua University Press. XXIX IAHR Congress, Beijing.

Bartone, E; Branstein, J; Eigen, J. (1994) "Urban Management Program toward Environmental Strategies for Cities", World Bank Technical Report.

Bhuiyan, A.A., (2001), "Urban Environment and Automobile Pollution- an Overview".

Binnie Partners (1994). "Preparation of structure plan, Master Plan, and Detailed Area Plan for Chittagong, component 2C: present situation report, traffic and transportation in Chittaging." Project BGD/88/052, UNDP/UNCHS(Habitat), Govt of Bangladesh.

Button, K. J. (1993). Transport Economics, 2nd Edition. Edward Elgar Publishing Limited.269 pp.

Environmental Pollution Report (1998), Department of Environment (DOE), Government of Bangladesh, Dhaka.

Firdus, G. M. (1984). A Study on the Problem of Bus Transit in Dhaka City. Master of Science thesis. Urban and Regional Planning. Bangladesh University of Engineering & Technology. Dhaka, Bangladesh.

Habib K.M.N (2002) "Evaluation of Planning Options to Alleviate Traffic Congestion and Resulting Air Pollution in Dhaka City." M.Sc. Thesis, Department of Civil Engineering, BUET, Dhaka.

Haworth.N(1995)"Road user behavior and safety in a developing country." Proc of the 7th World Conference on Trnasport Research, Sydney, July, 1995

Hoque, M.M (1997) Road safety Audit in Developing Countries. Research Report, Trsnsport Research Group, Department of Civil and Environmental Engg, University of Sothampton

Hoque, S. M. and Choudhury, C., 2009, "Mass Transit in Dhaka City", Workshop on Future Direction of Dhaka Urban Transport, Dhaka.

Hossain.M (2004), Shaping up of urban transport system of a developing metropolis in absence of proper management setup: the case of Dhaka Journal of Civil Engineering (IEB), 32 (1) (2004) 47-58.

Huzayyin A. S. and Osman O., 2001. Problems of Transport Management Institutional Development in the Developing Countries. in the proceedings of 9th WCTR, Seoul, 2001.

Iwasaki.M. (1992) “An analysisof capacity reduction on rural expressway”, Proc. of the !6th ARRB Conferernce, Part 5, Perth, Western Australia, 9-12 November 1992

Jaigirdar, M. A. (1998), “Environmental Impacts of Improved Bus Service of Dhaka City”, M.Sc. Thesis, Department of Civil Engineering, BUET, Dhaka.

Karim, M. M. (1998). Engineering, Regulatory, and Financial Options of Mass Transit in Metropolitan Dhaka. A report submitted to the Ministry of Communication, Bangladesh Government. 59 pp.

Kwakye, E. A., Fouracre, P. R. and Ofosu-Dorte, D. (1997). Developing Strategies to Meet
the Transport Needs of the Urban Poor in Ghana. World Transport Policy & Practice. Volume
3, No. 1. pp. 8-14.

Khan, M. A. (2007). When shall we get rid of Dhaka city traffic jam? Bitter Truth. The Daily Star, Bangladesh

Mahmud,K., Gope,K.,Chowdhury,S.M.R.C. (2012). Possible Cause & Solutions of Trffic Jam and their immact on the economoy of Dhaka city.

Oni S. I., 2001. Urbanization and Transportation Development in Metropolitan Lagos. in the proceedings of 9th WCTR, Seoul, 2001.

Population and Housing Census 2011 Bangladesh Bureau of Statistics Statistics and Informatics Division Ministry of Planning.

Rahman, K. A. (2001) Planning and Managing Eco-transport in Dhaka, Seminar on Ecotransport Assistance in Dhaka, Organized by Japan Transport Cooperation Association and Bangladesh Road Transport Authority, February 2001, Dhaka.

Stewart, R.D. (1975). “The Effects of Carbon Monoxide on Humans”, Annual review of Pharmacology, Vol. 15, pp.409-423.

STP, 2005, "Strategic Transport Plan for Dhaka", Prepared by Louis Berger Group and Bangladesh Consultant Ltd.

The Daily Star, 2010, online version, available at http://www.thedailystar.net/newDesign/news-details.php?nid=147756, Accessed on November, 2010.

Workshop on Urban Transport Development Strategy for Dhaka Metropolitan Area, January, 2001, GDTPCB, Dhaka City Corporation, Dhaka.

ROAD SAFETY ASSESSMENT FOR PEDESTRIANS ON DHAKA-ARICHA HIGHWAY (N5)

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ABSTRACT

The road safety status in Bangladesh is very serious and the rates of accidents are of the highest in the world. Among these crashes pedestrians are vulnerable road user group. Over half of all road deaths are caused by collisions between vehicles and pedestrians (WHO, 2009). It was estimated that 43 percent of all types of accidents are hit pedestrians was occurred in the Dhaka-Aricha highway (Hafez, 2012). In that case, analysis of fatalities, designing safe roads through road assessment to alleviate the consequence of the crash has become an important issue for road safety engineering. This study investigates the risk mapping and star ratings for pedestrians along the Dhaka-Aricha highway. Before conducting this study, different types of accident data have been analyzed (from 2000 to 2010) on this highway based on pedestrian's fatalities and those data were collected mostly from Accidents Research Institute (ARI) of BUET and other data were collected from Roads and Highways Department (RHD) and Bangladesh Road Transport Authority (BRTA). Among the total road length of 526.59km, only 27km have been analyzed (Amin Bazar to Dhamrai) and developed risk mapping and calculated road protection score along the road length and star ratings for pedestrians using iRAP methodology. The International Road Assessment Programme or iRAP assesses roads all over the world and aims to significantly reduce road casualties by improving the safety of road infrastructure. After a systematic analysis, we have found that only 2.59km road length at different locations (9 percent) were 3-stars and remaining length was observed as 2- stars rating. The star rating results reveal that pedestrian's safety situation of Dhaka- Aricha highway is at a risk. And it was found from the analysis that this situation is aggravating throughout the years (Hafez, 2012).

Keywords: *Pedestrian's fatality, iRAP, risk mapping ,star rating and road infrastructure*

1. INTRODUCTION

All over the world almost 1.2 million people are killed in road accidents every year, with about 88 percent of these accidents occurring in developing countries (WHO, 2009). In other statistics, every day more than 4,000 people die in road crashes all over the world and a significant numbers 137,000 (Source: iRAP) are injured and disabled. Road trauma is a serious and rapidly worsening public health crisis. Among these crushes pedestrians are the vulnerable road users.

In many countries, collisions with pedestrians are a leading cause of death and injury. In some countries, over half of all road deaths are caused by collisions between vehicles and pedestrians (WHO, 2009). In Bangladesh, the rate of fatalities of pedestrians are increasing at such a cautious rate that the scenario is not easeful at all.
In addition to the tremendous traumatic and emotional impact of road crashes, there is a vast economic consequence. It is estimated that road crashes cost the global economy US$1.4 billion every day, in terms of lost productivity, health care, emergency services. Crashes are leading cause of traffic congestion, itself a source of environmental damage.

Nationally, road crashes typically cost the equivalent of 1-3% of a country's Gross Domestic Product. Personally, road crashes can be the trigger that plunges a family into poverty. Road death and injury is not inevitable. Road trauma is a preventable public health challenge. Road systems can be developed that reduce the likelihood of a crash occurring and minimize the severity if a crash does occur. This can be achieved with safe road users understanding the risks they impose on themselves and on other road users, safe vehicles and safe roads, together with appropriate enforcement. In that case, analysis of fatalities, designing safe roads through road assessment to alleviate the consequence of the crash has become a highly sensitive issue for road safety engineering.

2. STUDY AREA

There are eight important national highways in Bangladesh. In this paper only a portion of N5(Dhaka- Aricha) has been focused as our study area. The Dhaka-Aricha highway plays a vital role in inter regional road transport in Bangladesh. This Highway is considered as man-made death trap. Major rehabilitation of this highway including safety improvement was undertaken at various times.

There are two very important roads in Bangladesh for national activities Dhaka-Aricha Highway is one of them. As a very important road, the standards of the road (roadway condition, roadway environment, level of service, etc) should be very high. But real scenario is not congruous according to the importance of the road. Nine divisions at savar as per their suitability maintain the roadway conditions.

The Dhaka-Aricha National Highway (N_5) has an overall length of 526.59 km. This national highway starts the journey from Dhaka (Mirpur Bridge/ Amin Bazar) and ends at Banglabandha 0 km post, in front of Panchagarh Rifels Sign Board. Amongst the whole roadway lenght of Dhaka-Aricha highway we have analyzed nearly 27km in length of road (up to Dhamrai, Noya Bazar) that shown in figure 1.

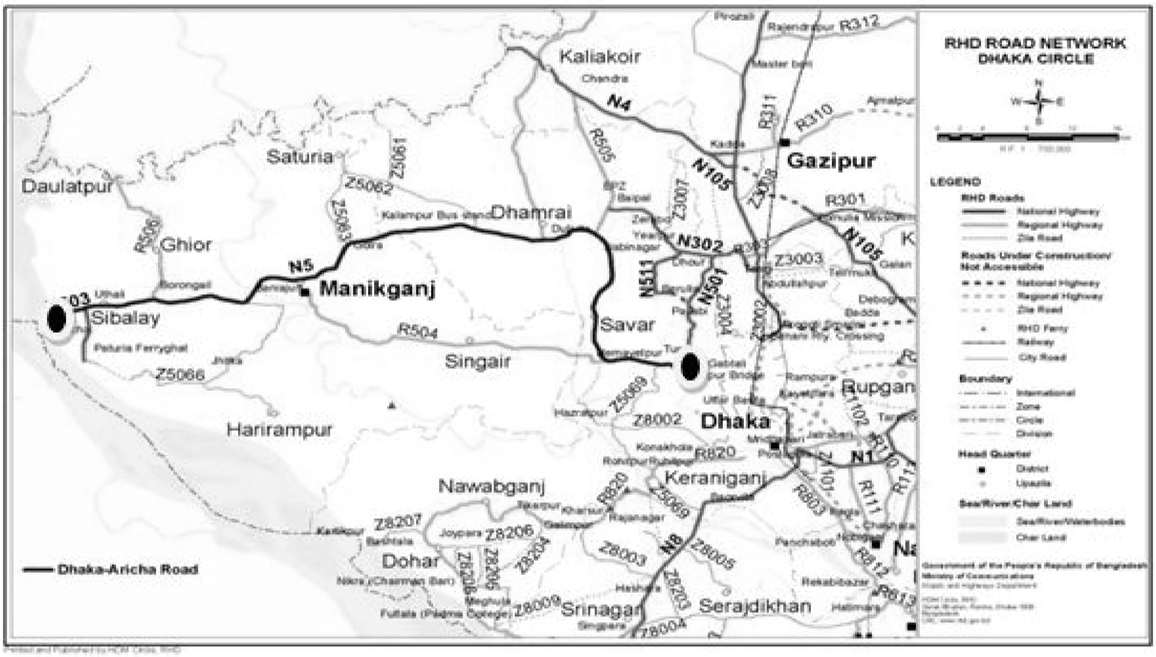

Figure 1: Study area (Amin Bazar to Dhamrai =27 km) in Map.

According to survey report of Road Maintenance and Management System (RMMS) of Roads and Highways Department (RHD) in the year 2008, the total single carriageway length is 508.78 km and total dual carriageway length is 17.81 km and shoulder width varies from 1.0 to 4 .0 meter with earthen shoulder at most of the locations.

3. METHODOLOGY

Preparation of Risk Map

The Road Assessment Programmes have developed maps to show individual risk (used to convey safety performance of roads to the public) and collective risk (used to convey safety performance to road safety professionals). At earlier in literature review risk mapping is discussed which actually rates the safety performance of the road based on the number of crashes that occur over a specific period and the amount of traffic using the road. Risk is categorized into following five colored banding:

- High risk- black color

- Medium high risk-red color
- Medium risk- orange color
- Low medium-yellow color
- Low risk-green color

★★★★★		Green color
★★★★		Yellow color
★★★		Orange color
★★		Red color
★		Black color

Figure 2: Star rating and color coding

Road Inspection

iRAP Star Ratings are based on a detailed visual inspection of a road's infrastructure elements. iRAP currently uses two types of road inspection:

- drive-through
- video-based.

The type of inspection conducted depends on the availability of technology, the complexity of the road network and the degree to which a project is focused on building the capacity of road safety stakeholder organizations. Part of the iRAP process includes the accreditation of those who will be conducting inspections. The accreditation relates both to the individuals who are conducting the process and to the equipment (vehicle camera, hardware and software) that is being used. Accreditation is tailored to the two inspection systems being used. In our case, a car was taken for the road inspection.

Road Inspection

RPS is to be calculated for the better understanding of the level of risk. Each road user RPS is the sum of RPS for the relevant crash types, which are in turn a function of likelihood, severity and crash-type calibration factors. The RPS is a unit-less measurement and is calculated for each road user for each 100 meter section of road. A high score equates with a high level of risk, and a low score equates with a low level of risk.

The structure of the pedestrian road protection score equation is shown in the following figure 3:

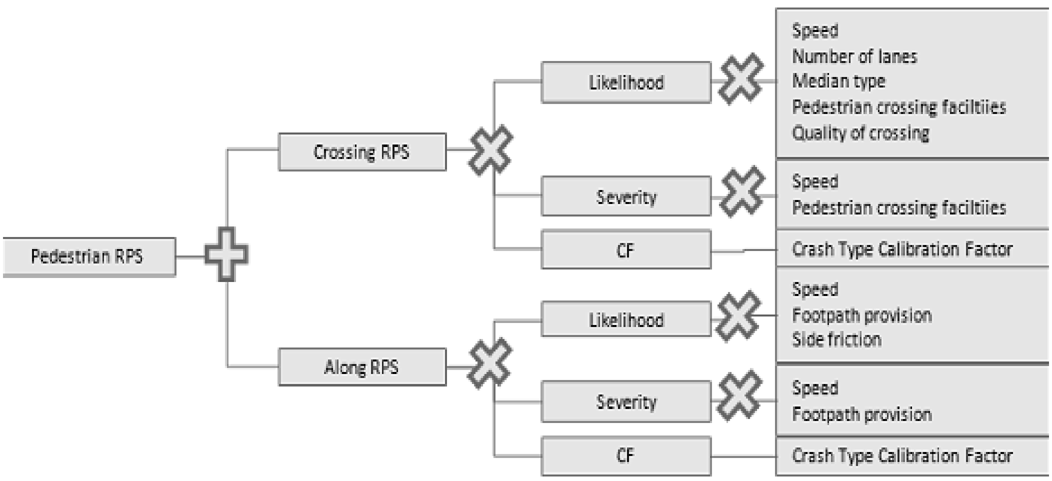

Figure 3: Pedestrian RPS equation. (Source: www.irap.org)

- **Data Processing**

After the collection of field data for star ratings different required attributes have been inputed into the iRAP tool kits for the processing of data, calculation of RPS and hence star ratings for pedestrians risk. The figures of the iRAP toolkits are given below for the better understanding.

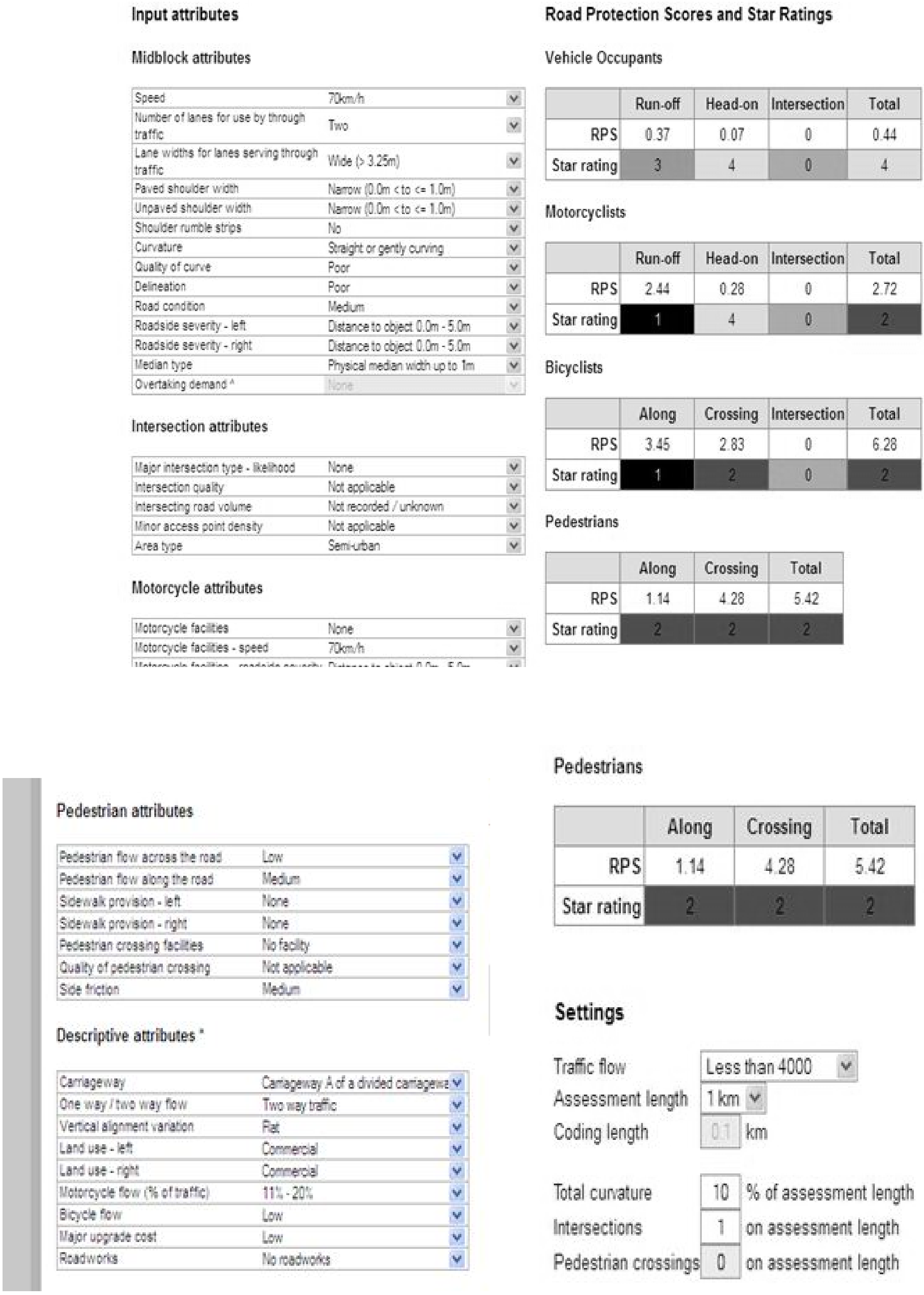

Figure 4: iRAP software for inputting attributes. (Source: www.irap.org)

4. RESULTS

For the better understanding of the star rating results for pedestrian, the combination of both risk mapping and road protection score at different section has been made. Obtained results are presented in this chapter both in tabular form and maps. Then the explanations of the star rating results are also established in remarks.

Results of Risk Mapping

The iRAP's first task is the risk mapping of the study area. For establishment of risk mapping the data of serious and fatal accidents and road design elements of Dhaka-Aricha National Highway has been analyzed and it has been found that the road section from Dhamrai Naya Bazar towards Amin Bazar with a length 2.59km at some locations is in a medium risk (as pedestrian flow is comparatively low) and rest of the length (24.41km) upto Amin Bazar bridge is in medium high risk (as pedestrian flow is extensive). This map represents that crash rates based on fatal and serious injuries per kilometer and per thousand vehicle, portraying risk and showing how risk changes as an individual moves from one road section to the next.

Table 1: Risk Intensity from Dhamrai to Amin Bazar Bridge.

Section (from – to)	Length (km)	Risk Intensity
Dhamrai Naya bazaar towards Amin Bazar bridge	2.59	Medium
Up to amin Bazar bridge	24.41	Medium High

Road Protection Score (RPS)

RPS scores is related to the relative risk which are calculated at different section of Dhaka-Aricha highway based on the data of traffic volume, average speed of motorized vehicles, lane and shoulder width. For each of those design elements, there is a certain amount of risk. RPS values are varying location to location of the highway. Average value of the RPS at different location is approximately 5 which denotes medium risk for pedestrians.

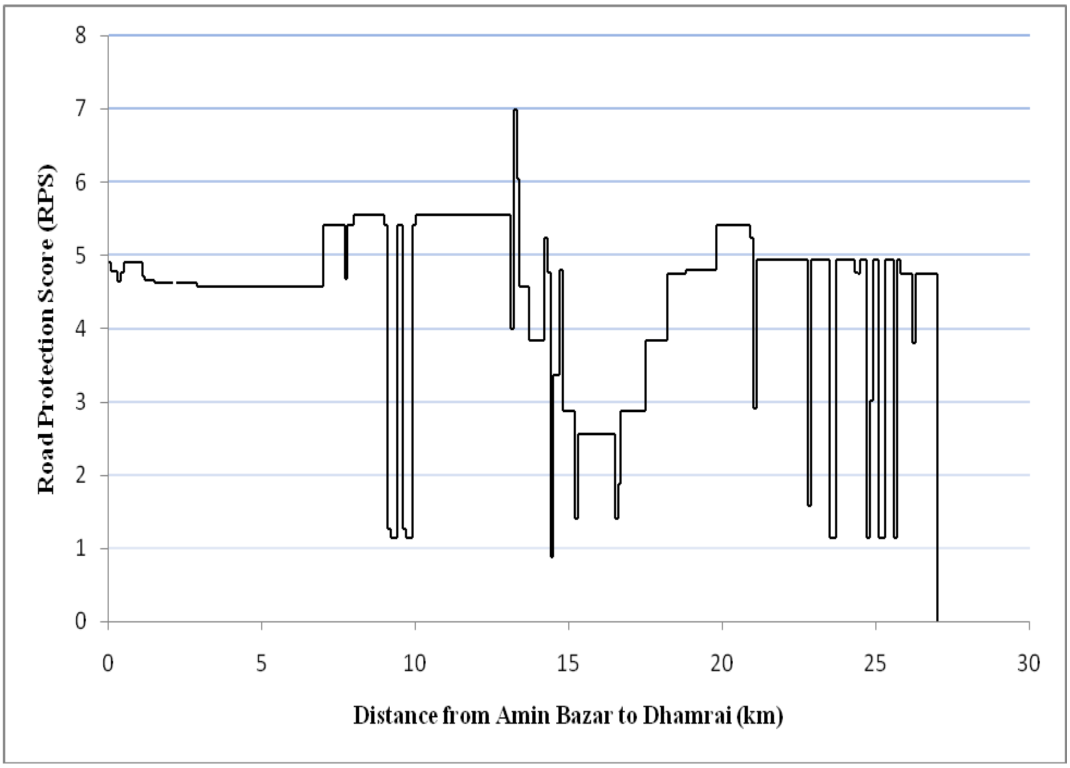

Figure 5: Road Protection Score for Pedestrian, N_5 (Amin bazaar to Dhamrai, 27km)

Pedestrian RPS Bandings			
Stars	From		To
			0
☆☆☆☆☆	0	to	0.4
☆☆☆☆	0.4	to	0.8
☆☆☆	0.8	to	4
☆☆	4	to	9
☆	Greater than		9

Figure 6: Relation between RPS and star ratings (www.irap.org)

Results of Star Ratings

According to the results of risk mapping, and RPS score it is clear that the relative risk for pedestrian along the route from Dhamrai to Amin Bazar is medium high except some locations where the risk is medium. From the data analysis it is obvious that pedestrian is the most vulnerable road user group along this highway. We have analyzed 27km roadway length amongst which only 2.59km road length at different section has rated 3-star and remaining length has rated 2-star. On the overall road network the rated percentage of 3-star road is 9.60 percent and for 2-star it is 90.40 percent. The star rating results is showed both tabular form and column diagram in following below.

Table 2: Pedestrian Star Rating (Dhamrai to Amin Bazar)

Length (km)	Percentage	Rated Star
2.59	9.60	3
24.41	90.40	2

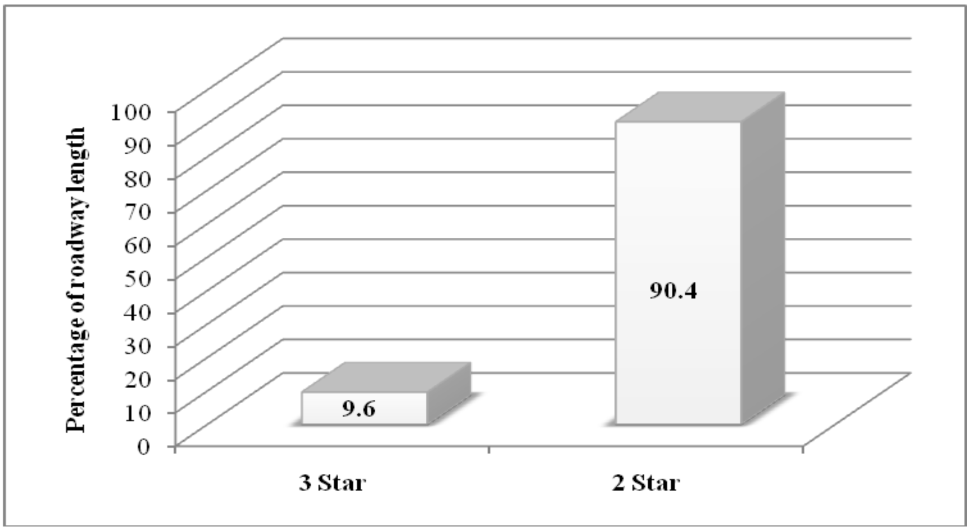

Figure 7: Percentage of roadway length of star rating of pedestrians

Risk Factors

There are several risk factors that influence the likelihood of occurring crash and its severity. These include behavioural factors of drivers, vehicle factors and road infrastructure elements. iRAP's primary focus is the road infrastructure elements risk factors.

Absence of separated footway and safety barrier for pedestrians, inadequate street lighting and traffic sign, no crossing and footpath facilities for pedestrians, and heterogeneous traffic flow in almost most of the cases of the Dhaka-Aricha highway was observed. The main risk factors for pedestrians are absence of crossing and walkway facilities in the Dhaka-Aricha highway.

RECOMMENDATIONS

From the analyses and results it is clear that pedestrian safety is at a risk for our selected study area and can be projected for rest of the roadway length. It might be the same or more aggravating scenario for the pedestrians' safety. At least, to hold the situation not to go further more worsening, some immediate physical measures where justified, engineering and educative measures should be taken without delay.

The present study shows the star rating for vulnerable road user group. This Valuable information can further be used for future Study regarding Star Rating for overall safety approach not only for pedestrians but also for whole systems.

5. CONCLUSIONS

In this paper, only a certain portion of Dhaka-Aricha highway has been investigated to measure the extent of risk of pedestrians and provided some recommendations to lessen the malignant scenario. Since data collection and star rating are time consuming and costlier processes, it was not possible to highlight the whole length of the highway. The main objective was to identify the risk and safety assessment for pedestrians and recommend taking necessary steps as immediately as possible to save lives and enhance the economic growth of the country. As the iRAP technique was proved effective to reduce pedestrian accidents in the developing countries like Bangladesh, proper actions should be taken urgently in a sustainable and systematic way. Otherwise, an unfortunate future is waiting for Bangladesh, road accidents will be the daily phenomena and death of people will be unavoidable situation.

ACKNOWLEDGEMENTS

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REFERENCES

Rahman, M. A. (2012),"Identification of Road Safety Measures on Dhaka-Aricha Highway using iRAP Star Ratings".

Hoque, M. M. (1991), "Accident Investigation for the Safety Improvement of Dhaka-Aricha Highway - A Section of Asian Highway", Final Report, Dept. of Civil Engineering, BUET, Dhaka, December 1991.

Hoque , S. M.,Moniruzzaman, S. M. S., and Mahmud. S. M. S.,(2010) "Effectiveness of black spot treatments along Dhaka-Aricha highway".

Hoque, M. M.,Smith, G.,Hossain. D. Z.,"Improving highway safety in Bangladesh: road improvement and the potential application of iRAP".

Website: http://www.irap.org

Website: http://toolkit.irap.org/

CAPACITY EVALUATION OF ROUNDABOUT INTERSECTIONS IN KHULNA METROPOLITAN CITY USING SIDRA

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ABSTRACT

The main emphasis of this study is to evaluate the capacity and Level of Service (LOS) of the roundabouts in Khulna Metropolitan City (KMC), Bangladesh. In KMC, increasing traffic volumes is the quickly developing major problem facing our modern society. These problems will continue and it may worsen in future due to rapid growth of population and vehicle numbers in KMC. Therefore, it is essential to evaluate the capacity of the roundabouts for proper traffic operation. This study is carried out to evaluate the capacities of the roundabouts in KMC. The capacity analysis is conducted by using the software SIDRA. For evaluating the capacity by using SIDRA, two types of data (geometric data and traffic volume data) were collected from the three roundabouts namely Royal Roundabout, Moilapota Roundabout, and Shibbari Roundabout in KMC. The necessary geometric data include number of circulatory lane, island diameter, circulatory roadway width, inscribed circle diameter, entry lane number, average lane width at entry, entry angle and entry radius as well as the traffic volume data were collected at peak periods for a period of one hour on working day of sunny weather with their direction of movements. From the analysis using SIDRA for the three roundabouts the degree of saturation (v/c) is found more than 0.85. This value is recommended by analysis procedure of some model countries such as Australia, Germany, United Kingdom and U.S.A., whose roundabouts are designed to operate at no more than 85 percent of their estimated capacity. The LOS is found F for the three roundabouts in KMC. Finally, the results show that the main causes of over saturation are the number of circulatory lane number, number of entry lane number and the high traffic volumes.

Keywords: *Roundabout, SIDRA, capacity, LOS, oversaturation.*

1. INTRODUCTION

The history of traffic circles is almost as long as that of signalized intersections. The first traffic circle concept was introduced in 1877 by French Architect Eugene Henard (De-Argao 1992). In 1903 he suggested that the traffic circle is a convenient form of traffic control when many roads converged. The concept of modern roundabouts was introduced in 1963 when the British government employed the off-side rule based on which the priority was given to the circulating vehicles on the traffic circles. For more than three decades modern roundabouts have been used successfully throughout the world as a junction control device (Akcelik, 1997). Evaluation of roundabout capacity is thus very important since it is directly related to delay, level of service, accident, operation cost and environmental issues.

In Khulna Metropolitan City (KMC) increasing traffic volumes and congestion are two quickly developing problems facing our modern society. It is common to see traffic congestion at junctions at peak hours in the morning and evening. Hence traffic police has to face difficulties to manage this growing traffic especially at peak periods. Poor road planning and sub-standard geometric conditions have a significant effect on roundabout capacity and traffic congestion (May, 1990). Otherwise, roundabout junction is more dependent on driver behavior and balanced traffic flow between the approaches.

These problems will continue and it may worsen in future due to rapid growth of population and vehicle numbers in KMC. As a consequence, traffic engineers are looking for new solutions to these problems. Therefore, it is essential to evaluate the capacity of the roundabouts for proper traffic operation. This study is carried out to evaluate the capacities of the roundabouts in KMC by using SIDRA.

2. METHODOLOGY

To achieve the objectives of this study, roundabout's traffic data at peak periods and geometric data were required. The geometric data should be measured correctly since geometric design will improve not only capacity but also safety, which is major concern for road design. Again traffic data should indicate the existing peak hour traffic conditions. The roundabouts were chosen based on the principle representative of the target population of the roundabouts in terms of size and numbers. The selected three roundabouts were Hotel Royal Roundabout, Moilapota Roundabout, and Shibbari Roundabout. All the roundabouts in KMC are more or less similar to each other, so these three roundabouts can represent all the roundabouts in the city.

2.1 Geometric Data Collection

According to the SIDRA software for capacity and delay analysis the geometric data collected include: number of circulatory lane, island diameter, circulatory roadway width, inscribed circle diameter, entry lane number, average lane width at entry, entry angle and entry radius. These data were measured with a tape and by using the concept of geometry.

2.2 Traffic Data Collection

The movement of traffic vehicles and their volume are important parameters in capacity analysis using SIDRA software. Thus traffic volume data were collected at peak periods for a period of one hour on working day of sunny weather with their direction of movements. Traffic volume was collected for each separated lanes. Traffic data were collected by using a video camera.

3. RESULTS AND DISCUSSION

As per the methodology, the geometric data collected include: number of circulatory lane, island diameter, circulatory roadway width, inscribed circle diameter, entry lane number, and average lane width at entry. These data were measured with tape. The collected geometric data are summarized in Table 1.

Table 1: Summary of Intersection Geometry

SI No.	Roundabout Name	No. of Legs	Number of Circulatory Lane	Island Diameter (m)	Circulatory Road Width (m)	Inscribed Circle Diameter (m)
1	Hotel Royal	3	2	6	9	15
2	Moilapota	4	2	7	11	18
3	Shibbari	4	3	19	17	36

From Table 1 it is observed that the island diameters of the roundabouts are 6m, 7m and 19m. When circulatory width is added, the diameters become 15m, 18m and 36m which can be categorized from mini roundabouts to multilane roundabouts according to Roundabout Information Guide (FHWA, 2000).

Another important parameters for this study are number of entry lanes, average lane width, entry angle and entry radius. Table 2 shows the summary of legs or approaches geometry.

Table 2: Summary of Legs or Approaches Geometry

SI No.	Roundabout Name	Leg Name	Number of Entry Lane	Average Lane Width (m)	Entry Angle (degree)	Entry Radius (m)
1	Hotel Royal	Khan Jahan Ali Road (From Ferrighat)	1	4.7	41	122
		KDA Avenue	2	6.4	28	3
		Khan Jahan Ali Road (From Rupsha)	1	4.7	51	17
2	Moilapota	KDA Avenue (From Shibbari Mor)	2	6.7	60	3
		Shere Bangla Road	1	3.8	33	8
		KDA Avenue (From Royal Mor)	2	4.2	21	4
		Shatkhira Road	1	4.6	55	9
3	Shibbari	KDA Avenue	2	5.9	71	9
		Ibrahim Mia Road	1	3.4	55	31
		Mojid Sharoni	2	3.4	53	3
		Khan A Sobur Road	2	7.9	51	13

In Table 2, it is found that the approach legs have maximum two lanes and the average lane width ranges between 3m to 8m.

The movement of traffic vehicles and their volume are important parameters in capacity analysis using SIDRA software. Traffic volume was collected for each separated lanes. The volume of each type of vehicles is summarized in Table 3.

Table 3: Vehicle Volume at Intersections at Peak Hour (4.30pm to 5.30pm)

Roundabout Name	**Heavy Vehicles**			**Light Vehicles**									**Total Traffic (PCU)**	**% of Heavy Vehicle**
	Bus	Truck	Total	Car	Pick up	Micro Bus	Jeep	Auto rickshaw & Easy bike	Van & Rickshaw	Motor cycle	Bicycle	Total		
Hotel Royal	41	35	76	128	31	61	15	1151	1525	779	743	4433	5332	4
Moilapota	24	35	59	153	22	32	16	807	2506	941	406	4883	6926	3
Shibbari	16	27	43	108	12	34	10	820	1187	272	166	2609	3569	4

From the Table 3 it is found that the percentage of heavy vehicles is not more than 4 percent and the light vehicles volume is larger than the heavy vehicles volume. The Moilapota roundabout has larger total traffic than other two roundabouts.

The traffic volume in passenger car unit (PCU) and the movement of the traffic on each approach leg are also essential for the analysis. The passenger car equivalent factors are used to convert the number of vehicles in PCU. The PCU values given in the Geometric design of Highways (MoC, 2001) are given in Table 4.

Table 4: PCU of Different Types of Vehicle in Bangladesh (MoC, 2001)

Vehicle Categories	**PCU**
Passenger Car	1.00
Light Goods Vehicle	1.00
Truck	3.00
Bus	3.00
Auto-rickshaw/Motorcycle	0.75
Rickshaw/Van	2.00
Bicycle	0.50

The summarized entry traffic flow on roundabout approach legs are shown in Table 5. From Table 5 it is found that the traffic flow is unbalanced at legs or approaches at the three roundabouts. However it is not recommended to design roundabout as traffic control devices when the traffic flow is unbalanced at different legs (FHWA, 2000).

Table 5: Summarized Entry Traffic Flow on Roundabout Approach Legs

SI No.	Roundabout Name	Leg Name	Entry Traffic on Legs (PCU)	Percentage of Traffic Share
1	Hotel Royal	Khan Jahan Ali Road (From Ferighat)	1282	24
		KDA Avenue	1718	32
		Khan Jahan Ali Road (From Rupsha)	2332	44
2	Moilapota	KDA Avenue (From Shibbari Mor)	3272	47
		Shere Bangla Road	882	13
		KDA Avenue (From Royal Mor)	1118	16
		Shatkhira Road	1654	24
3	Shibbari	KDA Avenue	790	22
		Ibrahim Mia Road	637	18
		Mojid Sharoni	915	26
		Khan A Sobur Road	1227	34

It has already mentioned that SIDRA software was used for the analysis. The summarized capacity analysis results are shown in the Table 6. The performance of the roundabouts is measured with the degree of saturation or v/c ratio and the level of service (LOS) is also applied according to US HCM.

Table 6: Summarized Capacity Analysis Results on the Intersections

SI No.	Roundabouts	Total Vehicle Flow (PCU)	Effective Capacity (vehicle/h)	Degree of Saturation (v/c)	Average Delay (sec)	LOS
1	Hotel Royal	5332	2044	2.861	522.4	F
2	Moilapota	6926	1581	4.613	957.9	F
3	Shibbari	3569	1840	2.042	246.4	F

From Table 6 it is found that the Moilapota roundabout has a very low effective capacity than the other two roundabouts and the Shibbari roundabout has the lowest degree of saturation compared to the others. The LOS of the considered three roundabouts has F.

Actually the capacity of the roundabouts depends on the performance of the approaches or legs and their v/c ratio is taken from the maximum v/c ratio of the approaches or legs. Figure 1 shows the peak flow or entry flow versus effective capacity.

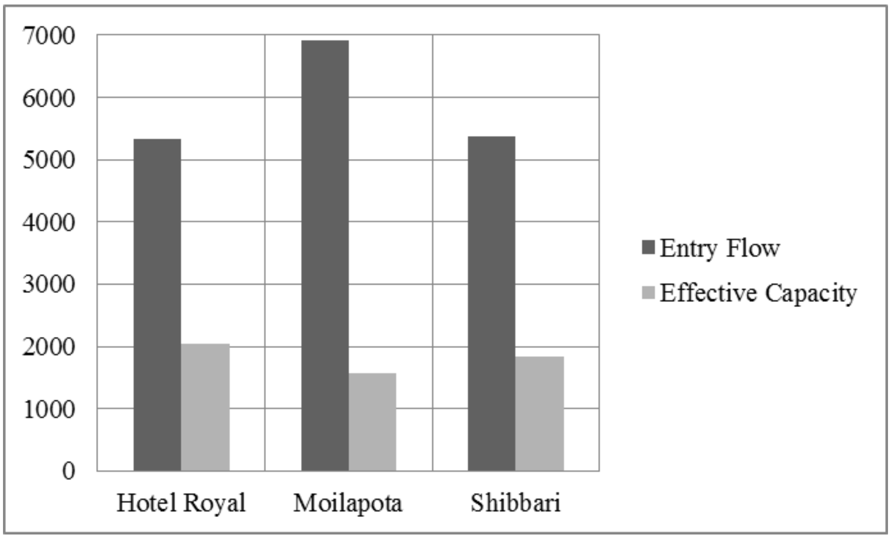

Figure 1: Graphical Representation of Peak Entry Flow versus Effective Capacity

For all the three roundabouts, lane by lane capacity has been carried out and capacity at legs, degree of saturation and opposing flow has been summarized. The summarized capacity analysis results on the approaches or legs are shown in the Table 7.

Table 7: Summarized Capacity Analysis Results on the Approaches or Legs

Roundabout Name	Leg Name	Entry Traffic on Legs (PCU)	Opposing Circulatory Flow	Degree of Saturation (v/c)	Capacity at Legs	v/c > 0.85
Hotel Royal	Khan Jahan Ali Road (From Ferighat)	1282	859	2.861	484	2.011
	KDA Avenue	1718	412	1.953	486	1.103
	Khan Jahan Ali Road (From Rupsha)	2332	83	1.960	1302	1.110
Moilapota	KDA Avenue (From Shibbari Mor)	3272	550	3.145	564	2.295
	Shere Bangla Road	882	1007	2.356	394	1.506
	KDA Avenue (From Royal Mor)	1118	553	1.104	537	0.254
	Shatkhira Road	1654	810	4.613	377	3.763
Shibbari	KDA Avenue	790	494	0.941	631	0.091
	Ibrahim Mia Road	637	533	2.042	633	1.192
	Mojid Sharoni	915	677	1.607	397	0.757
	Khan A Sobur Road	1227	999	0.744	453	-0.106

By observing the v/c >0.85 column of Table 7, which is based on US HCM (TRB, 2000), it is easily identify the legs which are in critical condition. It is seen that v/c ratio of the mentioned seven legs at Hotel Royal, Moilapota and Shibbari roundabouts are greater than 0.85. So, these legs are in critical condition. The problem of the approach is identified by using Table 7 which shows v/c>0.85, traffic volume of entry flow at legs and traffic volume of circulatory flow. Table 8 shows the summary of the conditions of roundabouts problems.

Table 8: Summary of the Condition of the Roundabouts

SI No.	Roundabout Name	Leg Name	Problems
1	Hotel Royal	Khan Jahan Ali Road (From Ferighat)	Entry lane number is not adequate and circulatory lane number is also not adequate.
		Khan Jahan Ali Road (From Rupsha)	Entry lane number is not adequate.
		KDA Avenue	Entry lane number is not adequate.
2	Moilapota	Shatkhira Road	Entry lane number is not adequate and circulatory lane number is not adequate.
		Shere Bangla Road	Circulatory lane number is not adequate.
		KDA Avenue (From Shibbari Mor)	Entry lane number is not adequate and circulatory lane number is also not adequate.
3	Shibbari	Ibrahim Mia Road	Circulatory lane number is not adequate.

4. CONCLUSIONS

From the results of the capacity analysis of the roundabouts in Khulna Metropolitan City (KMC), it is found that most of the roundabouts are in serious problems. Over saturation is the most common problem of them. Based on the observed field conditions, it is common to detect that at the peak hours the traffic police have to regulate the traffic at these roundabouts. The study showed that the major problems are related to the number of entry lanes, number of circulatory lanes, high traffic flow and unbalanced traffic on the approaches which in fact, not recommended on the roundabouts. Besides, the roundabouts are built when the traffic flow was lower and without considering future traffic extension. Even certain most important geometric elements (such as deflection, proper island splitters, etc.) are absent in the roundabouts of the Khulna Metropolitan City namely Hotel Royal roundabout, Moilapota roundabout and Shibbari roundabout.

5. RECOMMENDATIONS

It is recommended to revise the geometric data of the roundabouts and to build up the essential geometric elements properly as stated in the design manual of modern roundabout since they are very helpful to have reasonable capacity and traffic safety. The collected traffic data of the roundabouts at peak hours have high and unbalanced traffic flow and many legs of the roundabouts are found to be over saturated. It is recommended to increase the number of entry lanes, the number of circulatory lanes and the width the entry lanes. In this study, pedestrian traffic volume cannot be considered because of the time limitations. So it is recommended to include pedestrian traffic volume in the capacity analysis of the roundabouts by using SIDRA since it affects greatly the normal traffic flows and the capacity of the roundabouts.

6. REFERENCES

Akcelik, R. (1997). "Lane by Lane Modeling of Unequal Land Use and Flares at Roundabout and Signalized Intersection", the SIDRA solution, Traffic Engineering & Control, Volume 38, No. 7/8, Australia.

DeArgao, P. (1992). "Circles and roundabouts: A historical review, Institute of Transportation and Planning", Swiss Federal Institute of Technology, Lausanne, Switzerland.

FHWA (2000). "Roundabouts: An Information Guide Available at the Turner-Fairbank Highway Research Center".

May, A. D. (1999). "Traffic Flow Fundamentals", New Jersey, USA.

Ministry of Communication (MoC) (2001). "Geometric Design Standard of Roads & Highways Department", Government of the People's Republic of Bangladesh, Bangladesh.

Transportation Research Record (TRB) (2000). "Highway Capacity Manual", National Research Council, Washington D.C., USA.

RAIL SAFETY AT LEVEL CROSSINGS IN BANGLADESH

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ABSTRACT

The study analyses rail accident data of six years (2005-06 to 2010-11) collected from Chief Operating Superintendent department of Bangladesh Railway. Level crossing collisions are the major type of train collisions based on fatalities. Data revealed that 73 percent of total train collisions occurred at level crossings that contributed *90 percent of total deaths. Level crossing collision rate per million train-km was 2.7. Level crossing collision severity index (CSI) has been calculated as the number of people killed per 100 collisions with a value of nearly 73. Level crossing collision fatality rate was 1.94, defined as the number of deaths per million-train km. The violation of road traffic rules by road users at level crossings, unauthorized access to the rail tracks or crossing and shortage of required gateman are the main impediments to provide safe and efficient level crossing operation. Installation of barriers, construction of overpasses and underpasses at crossings with dense traffic, reduction of the number of level crossings, improvement of conditions such as visibility at crossings along with public awareness and appropriate enforcement actions.*

Keywords: *Level Crossing, Road and rail users, Collision, Collision Severity Index, Fatality rate.*

1. INTRODUCTION

Level crossings are well known as a component of railway networks with the greatest risk of collision and possibly derailment (Silmon, 2009). Level crossing collisions are the serious type rail accidents dealing with at least two transport modes (Siti, 2007). Rail-road intersections are very unique, special, and potentially dangerous and yet unavoidable. Road users and railway users are directly involved in this complex system. Here two different entities with entirely different responsibilities, domains, performances come together and converge for a single cause of providing a facility to the road user. During the normal operation also, there is every possibility of accidents occurring even with very little negligence in procedure and the result is of very high risk (Konkan Railway). That's why; level crossing safety is the key issue for the railway safety. No railway system can survive by ignoring this vital aspect. But in Bangladesh, most of the level crossings are unguarded and unprotected. This triggers frequent number of level crossing collisions every year. So it needs highly attention on rail safety at level crossings in Bangladesh.

2. LEVEL CROSSING COLLISION

To pass the vehicular traffic across a railway track, crossings are provided on the railway lines. When the road traffic passes at the same level as that level of the railway track, the crossing is termed as level crossing (Agarwal, 2007; www.railsafety.co.nz).
Level crossing collisions are one of the most severe rail accidents which have the potential to injure railway staff, passengers, road users and pedestrians (Huang, 2006; MOTC, 1998).

In Bangladesh the following types of rail accident are common. To divide the all rail accidents into two major groups (Collisions and Derailments), the accident at level crossings are included in Collision group as level crossing collision.

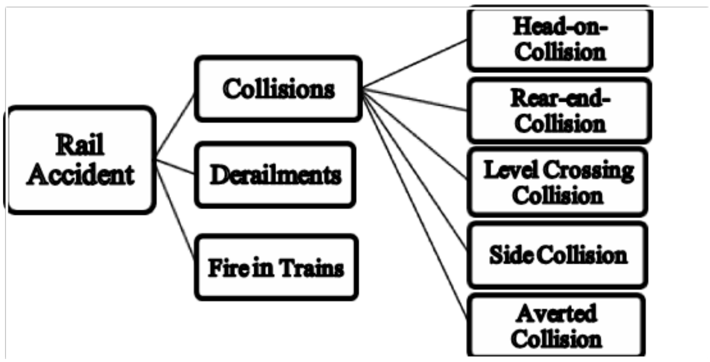

Fig. 1: Types of Rail Accident

3. LEVEL CROSSING COLLISIONS FREQUENCY ANALYSIS

3.1 Magnitude Of Level Crossing Collision

Table 1 shows that during the analysis period, a total 344 numbers of collisions occurred of which 252 were level crossing collisions. The percentages of different types of collision are graphically represented in the fig. 2. It is seen that level crossing collisions were the highest amount (73%).

Table 1: Year wise statistics of train collisions (Bangladesh Railway, 2009; Bangladesh Railway, 2010)

Type	Year (July-June)	Total	2005-06	2006-07	2007-08	2008-09	2009-10	2010-11
Train Collisions	Averted CL	63	10	10	9	12	8	14
	Level Crossing CL	252	40	42	53	47	43	27
	Head on CL	16	2	2	1	5	3	3
	Rear end CL	6	0	1	1	1	2	1
	Side CL	7	1	0	4	1	1	0
	Total	344	53	55	68	66	57	45

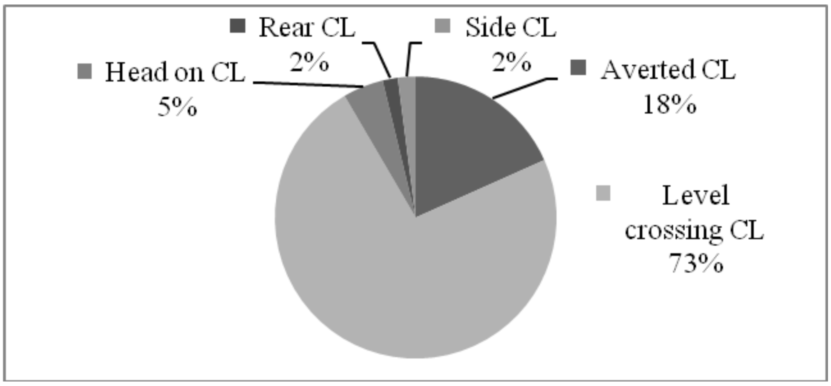

Fig. 2: Distribution of different types of collision

3.2 Level Crossing Collision Rate Per Million Train-Km

It is seen from table 2 that within six years study period the rate of level crossing collision was in an upward trend in the first three years but after 2007-08, rate gradually decreased. The average rate of level crossing collisions was 2.7 per million train km in the period from 2005-06 to 2010-11. This rate was 0.74 during the seven years from 1988 to 1995 (ESCAP, 2000).

Table 2: Level crossing collisions per million train-km

Year	Total Train-Km	No. of Collisions at LC	LCC per Million Train-Km
2005-2006	15229000	40	2.63
2006-2007	15141000	42	2.77
2007-2008	15558000	53	3.41
2008-2009	15703000	48	3.06
2009-2010	15805000	43	2.72
2010-2011	15805000	26	1.65
Average			2.7

Source: Department of COPS, Bangladesh Railway

4. LEVEL CROSSING COLLISIONS CASUALTY ANALYSIS

4.1 Casualties statistics

It is found from table 3 that during the study periods, 443 people were injured and 200 people were died by total train collisions. It is also seen that level crossing collisions were the most hazardous collision. Because, among the total injuries and total fatalities due to collisions, 76% injuries and 90% fatalities were due to level crossing collisions. Head-on-collision was the second position.

Table 3: Year wise casualties

Year (July-June)		Total		2005-06		2006-07		2007-08		2008-09		2009-10		2010-11	
Casualties		Injured	Killed	Injured	Killed	Injured	Killed	Injured	Killed	Injured	Killed	Injured	Killed	Injured	Killed
Collisions	Averted CL	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	Level Crossing CL	337 (76%)	180 (90%)	53	20	92	49	47	41	70	31	24	14	51	25
	Head on CL	93 (21%)	12 (6%)	7	0	0	0	0	0	32	3	1	0	53	9
	Rear end CL	13 (3%)	8 (4%)	0	0	0	0	9	8	4	0	0	0	0	0
	Side CL	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	Total	443	200	60	20	92	49	56	49	106	34	25	14	104	34

Source: Department of COPS, Bangladesh Railway

4.2 Level Crossing Collision Severity Index

The collision severity index (CSI) measures the seriousness of a collision. It is defined as the number of people killed per 100 accidents (Ramesh, 2011). Table 4 shows the year wise level crossing collision severity index. From the fig. 3, it is found that the CSI value was high in the year 2006-07 then decreased gradually in the next three consecutive years and reached to its minimum value of 32.56 in 2009-10. Again in 2010-11 the CSI increased nearly three times than previous year.

Table 4: Level crossing collision severity index

YEAR (1)	No. of Persons killed by LCC (2)	Total No. of LC Collisions (3)	CSI (4) = (2/3)*100
2005-2006	20	40	50.00
2006-2007	49	42	116.67
2007-2008	41	53	77.36
2008-2009	31	48	64.58
2009-2010	14	43	32.56
2010-2011	25	26	96.15
Average			72.89

Source: Department of COPS, Bangladesh Railway

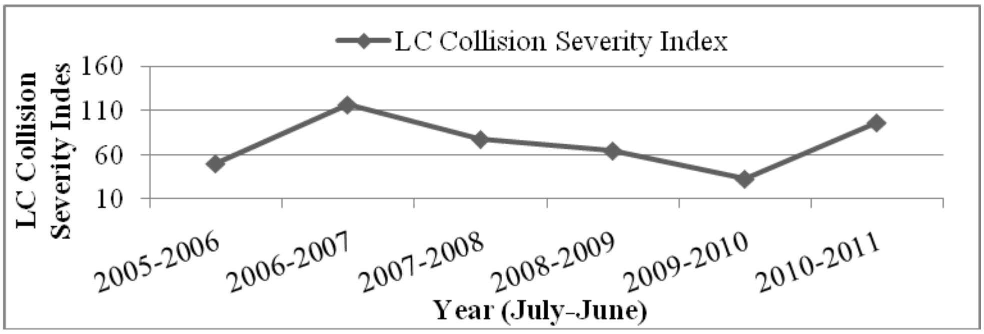

Fig. 3: Level crossing collision severity index (People killed per 100 collisions)

4.3 Level Crossing Collision Risk

In the table 5, the risk characterized by equivalent fatality, due to collisions is calculated and the risk diagram is highlighted in the figure 4. It is seen that collision risk diagram has fluctuated throughout the study periods. At the initial period the value of collision risk was 40, which increased almost double in next year and reached to its peak. Then the value decreased but remained more or less same in the next two years. Surprisingly this collision risk was too little in 2009-10 which rose up in 2010-11 almost double compare to the previous year.

Table 5: Level crossing collision risk

YEAR (1)	No. of Persons killed by LCC (2)	No. of Persons Injured by LCC (3)	Total No. of LC Collisions (4)	Equivalent Injury (5)= 3*(2)+(3)	LC Collision Risk / Equivalent fatality (6)= (2)+ 0.368*(3)
2005-2006	20	53	40	113	40
2006-2007	49	92	42	239	83
2007-2008	41	47	53	170	58
2008-2009	31	70	48	163	57
2009-2010	14	24	43	66	23
2010-2011	25	51	26	126	44

Source: Department of COPS, Bangladesh Railway

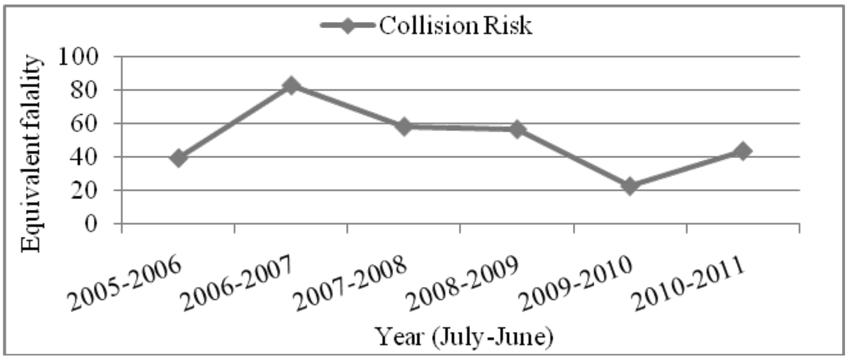

Fig. 4**:** Level crossing **c**ollision risk diagram (Equivalent fatality per year)

4.4 Level Crossing Collision Fatality Rate

Table 6 represents the fatality rate of the level crossing collisions for six years period. The collision fatality rate is defined as the number of accidental death per 10,000 trains run (Ramesh, 2011). In the figure 5, it is found that there was an increase in fatality rate from 1.90 in 2005-06 to 4.84 in the next year although less number of trains counted 4188 ran in that year. After 2006-07, it decreased gradually up to 2009-10 but the fatality rate again increased in the last year.

Table 6: Level crossing collision fatality rate

YEAR (1)	Total Trains run (2)	No. of Persons killed by LCC (3)	LC Collision Fatality Rate (4)=(3)/(2)*10000
2005-2006	105497	20	1.90
2006-2007	101309	49	4.84
2007-2008	107749	41	3.81
2008-2009	108203	31	2.86
2009-2010	107310	14	1.30
2010-2011	107310	25	2.33
Average			2.84

Source: Department of COPS, Bangladesh Railway

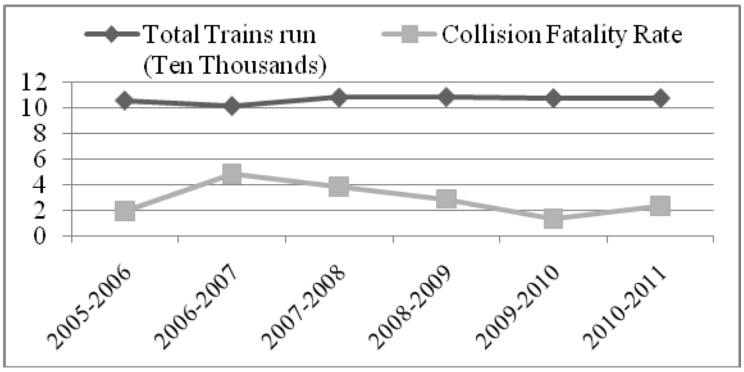

Fig. 5: Pattern of LC fatality rate

5. VEHICLES INVOLVEMENT AT LEVEL CROSSING COLLISIONS

Vehicles involvement means which vehicles are involved with level crossing collisions. Fig. 6 shows that the involvement of different types of road vehicles in level crossing collisions. It is found that the involvement of truck, covered van, trolley and power tiller results the almost half (49%) of the total level crossing collisions which was four times more than bus involvement. On the other hand, fig. 7 reveals that maximum percentage of deaths occurred (43%) from bus and train collisions at level crossings. Three wheelers such as votvoti, nosimun, mishuk, CNG, baby taxi and auto rickshaw are the most vulnerable to level crossing collisions, because the proportion of involvement and death for 3-wheelers in the level crossing collisions were the same percentage which was 8%.

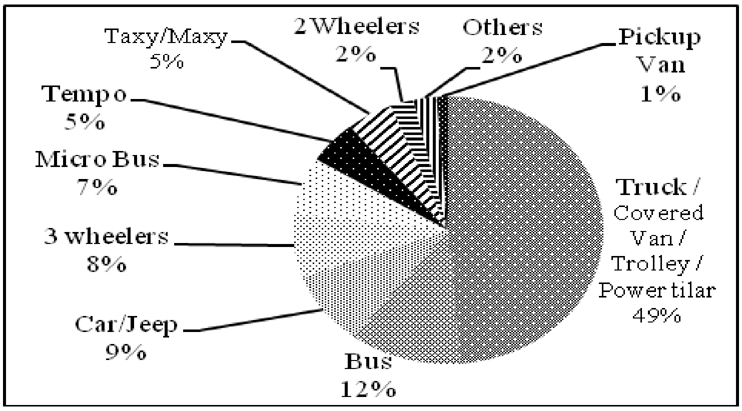

Fig. 6: Vehicles involved in level crossing collisions

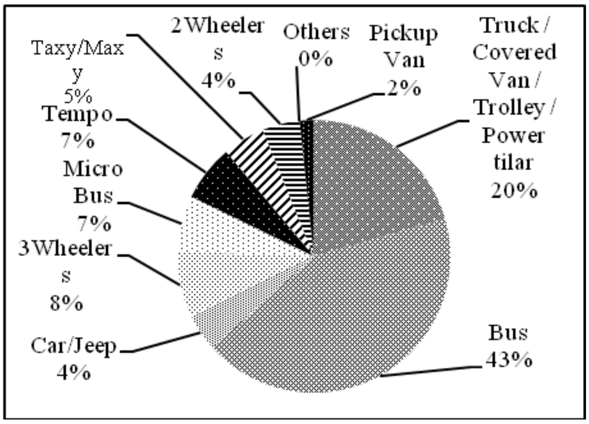

Fig. 7: Deaths by types of vehicles in level crossing collisions

6. CAUSES OF LEVEL CROSSING COLLISION

It is observed from the table 7 that people are responsible for almost 99 % LC collisions. Study shows that the violation of road traffic rules at railway-highway grade crossing by road users is the main cause for the level crossing collisions.

Table 7: Causes of level crossing collisions

Types of Collision	Causes of Collisions	No.of Repetition	Responsible people	Injured	Killed
Collision at Level Crossing	Careless train operation	1	LM & ALM		
	Pumping due to heavy load	1	Track Failure		
	Signal given without closing LCG & Traffic rules violation by Container Lory	1	Gate man & Road User		
	Traffic rules violation by Driver of different types road vehicle	248	Road Users	337	180
	Under Investigation	1			

Note: LM= Loco Master, ALM=Assistant Loco Master

Moreover unguarded, unprotected, unauthorized, and busy level crossing without safety measures is one of the key factors for level crossing collision. In Bangladesh, a large number of the gatemen are engaged on temporary basis or daily basis. That's why they are not serious about their duty. Illegal level crossings are building up here and there. Two sample pictures are shown in the fig. 8.

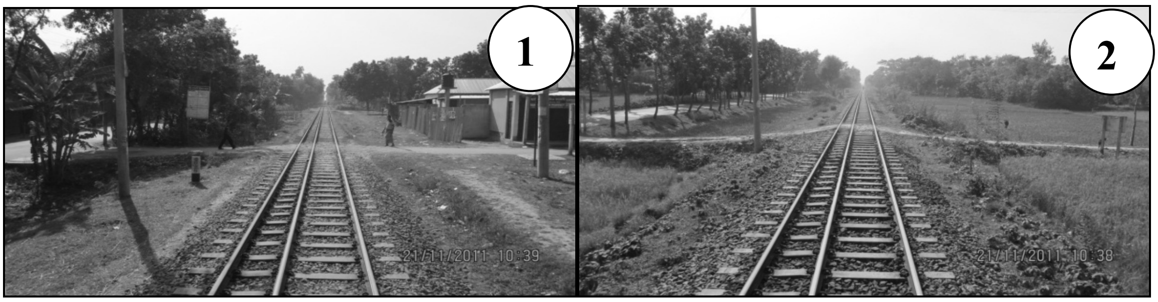

Fig. 8: (1), (2) Unauthorized level crossing of joydebpur-mymensingh route

In addition, there are 1184 numbers of authorized level crossings among 1523 without any gatekeeper which is 80% (Bangladesh Railway (BR),2008). Even no such arrangements are made, for example gate, barrier, road signs or road markings hence, there is danger of accident between the vehicles on the road and moving train. These haphazard situations are highlighted in the pictures below.

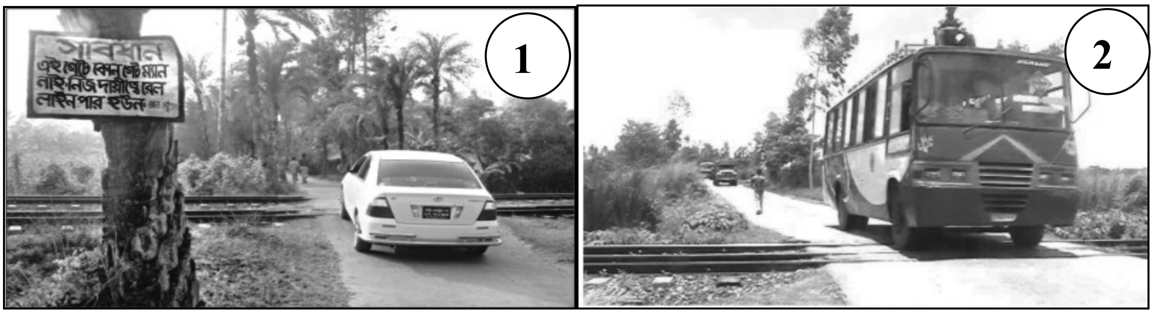

Fig. 9: (1), (2) Authorized unguarded level crossing (The Daily Star, 2010)

7. DENSITY OF LEVEL CROSSING

In Bangladesh, level crossing density is 0.83 per route km based on the preserved data of 2008. Bangladesh Railway had a total of 2835 route kilometers at the end of the year 2008-2009 and a total of 2340 level crossings (Bangladesh Railway, 2009; Bangladesh Railway, 2010).
Moreover, in Bangladesh, level crossing installation and upgrading priorities are established on the basis of the assumed road and rail traffic volume likely to use crossings in future. Bangladesh Railway does not take counts of road traffic and consequently road traffic density at individual crossing is known only from local experience (ESCAP, 2000).

8. CONCLUDING REMARKS

Level crossings of Bangladesh are the most vulnerable to train collisions. Among the types of train collisions, the proportions of level crossing collisions were predominant. The violation of traffic rules by road users at rail-road intersections is a common phenomenon in Bangladesh due to lack of safety awareness of the mass people. Moreover most of the collision prone level crossings were unmanned and lacked safety protections such as gate, barrier as well as road sign, road marking, warning signals etc.

Though the number of occurrence of level crossing collision is not too high with respect to other rail accident such as derailments, the casualties due to level crossing collisions become significant issues for Bangladesh. It is revealed from the analysis that the rate of level crossing collision is decreasing during the last 4 years of study periods which designates a positive remark about collisions. However the risk and severity index of level crossing collisions gets the attention again due to its increasing trend in the last year.

Grade separation is the best solution and undeniably the most effective means to eliminate danger at level crossings. Since non availability of reliable power supply at all Level Crossings is another constraint to go for automatic level crossing protection system in Bangladesh, railway managements should adopt a policy of manning currently unprotected crossings and equipping them with inexpensive locally manufactured barrier and warning systems. Bangladesh railways should carry out intensive social awareness, on a regular basis, to educate road users, villagers and motor vehicle drivers and make them aware of the provisions of motor vehicles act and railway act. Beside this appropriate enforcement action and penalties should be taken against people who ignore level crossing warnings.

9. REFERENCES

Agarwal, M.M., (2007), "Indian Railway Track", Design, Construction, Maintenance & Modernization, 15th Revised & enlarged edition.

Bangladesh Railway (BR), (2008), "A book of Level Crossing List, 2008".

Bangladesh Railway (BR), (2009), "Information book, 2009".

Bangladesh Railway(BR), (2010), "Information book, 2010".

ESCAP, (2000), "Evaluation of Cost-Effective Systems for Railway Level-Crossing Protection", Economic and Social Commission for Asia and the Pacific, United Nations, New York, 2000.

Huang W.S., (2006), "Exploring the Factors Affecting the Level Crossing Accident of TRA", National Chiao Tung University, 2006.

Konkan Railway, "Cost Effective System for Railway Level Crossing Protection", Konkan Railway CorporationLimited, (A Government of India Undertaking), CBD Belapur, Navi Mumbai – 400 614.

Ministry of Transportation and Communications (MOTC), (1998), "A study on the planning of railway level crossing safety improvement in Taiwan", MOTC, Taiwan, 1998.

Ramesh, A. and Kumar, M., (2011), 'Road Accident Models for Hyderabad Metropolitan City of India', Indian Highways, Vol. 39, No.7, 2011.

Silmon J. and Robertsa C., (2009), "Using functional analysis to determine the requirements for changes to critical systems: railway level crossing case study", Centre for Railway Research and Education, GisbertKapp Building, University of Birmingham, Birmingham B152TT, United Kingdom, September 14, 2009.

Siti Z.,(2007), "The Development of Railway Level Crossing Safety Assessment Model: A Research Framework", 29th Conference of Australian Institutes of Transport Research (CAITR), University of South Australia, Adelaide, Australia, 5th – 7th December 2007.

www.railsafety.co.nz (2013), Rail Safety, Newzealand.

SIGNIFICANCE OF DERAILMENTS IN RAIL SAFETY IN BANGLADESH

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ABSTRACT

In Bangladesh, throughout the past several years a large number of train accidents occurred of which most were train derailments. A total of 4010 numbers rail accidents took place during 2005-10 among which 86% were train derailments. Of these 73% were the consequences of the failure and weakness of infrastructure elements. Operational elements share of 16%, rolling-stock elements contribute 7% and the rest took place due to the combination of two or three elements. Branch lines were in more dilapidated state comparative to main lines with 46% on branch line, while 20.6% on main line, 8.1% on loop line and remaining percentage at yard. Meter gauge (MG) track are more derailment prone than broad gauge (BG). About 61% derailments occurred in passenger trains, 5% on freight train and remaining 34% on others train (reck, shunting, locomotive & miscellaneous). Average block time was 7 hours per derailment in BR which proves that a derailment. Derailment intensity map shows that Mymensingh-Shambhuganj and Rajapur-Sadar Rasulpur sections were highly derailment prone in case of main line while Hatibandha-Votmari and Netrokona-Thakurakona sections among branch lines. Main causes of frequent derailments at these sections were the insufficient strength of the track due to inadequate track components.

Keywords: *Bangladesh Railway, Train derailments, Rail infrastructure, Rolling stocks, Derailment intensity map.*

1. INTRODUCTION

The provision of safe and reliable services is a fundamental requirement of the railway. Passengers are entitled to expect to travel in safety and on time where Staffs are entitled to work in safe conditions (Delivering a Sustainable Transport,2007). Thus, Safety can be stated as the timely transit of passengers and goods without risk of casualties and damage which is basically the product of good practices at all levels of functioning i.e. design, manufacturing, maintenance and operations (Amitabh,2004). Though, railway transportation system is recognized as a safest mode of land transport around the world, railway accident like derailment has always been the major challenge for rail safety as well as a point of attention to the engineers and researchers (Agarwal,2005; Brabie,2005; H. Wu and N. Wilson.,2006).

In Bangladesh, though railway is considered as the safe mode of land transport of the country, rail network still not connected with all the major districts and cities. Beside, existing railway route length is decreasing instead of increasing (Bangladesh Railway (BR) website,2011). In addition, both passengers and freight movement were decreased continuously to a substantial amount for about thirty years after liberation and trying to revive its position for passenger service in the most recent years (Bangladesh Railway,2009). Furthermore, Bangladesh Railway (BR) has been experiencing a large number of train accidents since its development in 1972. The total number of accidents from 2001-2010 was approximately 5938. Among these, most of the accidents were train derailments (Bangladesh Railway,2009; Bangladesh Railway,2010). From 2001-2010, about 5461 train derailments were taken place, which is approximately 92% of the total incidence. Disruption to services, substantial financial losses and safety risk to staff and passengers are highly obvious from such occurrence (Barkan,2003).

After all, no research work has been carried out for Bangladesh railway regarding train accidents to determine the underlying causes for the development of effective countermeasures to enhance railway safety. Hence, the main objective of this work is to represent the train derailments scenario of Bangladesh railway including various statistics, factors and causes of derailments etc. to understand the significance of derailments in rail safety.

2. TRAIN ACCIDENTS AND DERAILMENT

2.1 Train Accidents

A train accident is termed as any occurrence which does or may affect the safety of the railways, its engine, rolling stock, permanent way, works, passengers or servants which either does or may cause delays to trains or loss to the railways (Arora,2006).

2.2 Train Derailment

Train derailment is a phenomenon or an accident in which the rail vehicle goes off the track while running, in a part or whole due to the instability of rail-wheel forces resulting from defects in operation, vehicle, track and sometimes due to the natural forces such as wind, seismic and landslides (Agarwal,2007; Wu H.,1999; Wikipedia,2011).

2.3 Types Of Train Derailment

Train Derailments are classified into two broad categories: (i) Sudden train derailment and (ii) Gradual or flange climbing train derailment. Sudden derailments means when the wheel sets jumping off the track suddenly. Such a derailment indicates that the derailing forces were high enough to suddenly force the wheel off the rails. The probable causes of sudden derailment are easier to find compare with gradual derailment (Arora,2006; Mundrey,2010). On the other hand, Gradual derailment occurs by flange climbing i.e. by wheel mounting the rail in a relatively gradual manner. It indicates that derailing forces were powerful enough to overcome the normal stabilizing force, yet not sufficient to cause a sudden derailments (Arora,2006; Blader,1990).

3. RAILWAY SAFETY

Around the world, railways have come to be recognized as a safest mode of mass transportation because of its inherent characteristics. Therefore, safety becomes the foremost issue while transporting man and materials in railway system. To ensure railway safety around the world all the activities includes according to the three-tier approach as follows (Rail Safety New Zealand,2011).

a) **Education:** includes (i) Advertising, (ii) Publicity and media relations, (iii) Awareness raising events and campaign, (iv) Development of education resources for schools (v) Publication and display of rail safety pamphlets brochures (vi) Training of etc.
b) **Engineering:** includes (i) Ensuring structural and functional integrity of the infrastructure and its subsystem (Track improvements, periodic maintenance, level crossing up gradation, Enhanced track inspection technology etc.) (ii) Ensuring structural and functional integrity of the rolling stock, (iii) Ensuring appropriate operational procedures and information management for effective train handling etc. (improved signaling and interlocking system).
c) **Enforcement:** includes (i) Application of appropriate warnings or prosecution against those who fail to obey the rules and regulations.

4. STUDY APPROACH

The main objective of this study is to represent train derailment accidents thus to established the overall scenario of train derailments in Bangladesh Railway. In this regards accidents data consists of six consecutive years (2005 to 2010) collected from the department of COPS (Chief of Operating Superintended) of Bangladesh Railway has been analyzed in a number of ways based on the prominent aspects as follows:

(a) Magnitude, trend and rate of derailments
(b) Distribution of derailments by elements & causes, track gauge, line types and train types.
(c) Average block time per derailment
(d) Identification of derailment prone locations.

(E) ANALYSIS OF TRAIN DERAILMENTS

4.1 Magnitudes Of Train Derailments

Analysis of train accidents data shows that there were about 4010 nos. of train accidents took place in Bangladesh railway among which 3464 nos. or 86.4% were derailments and 13.6% were others train accidents including collisions, fire in trains and train run into obstructions (see fig.1).

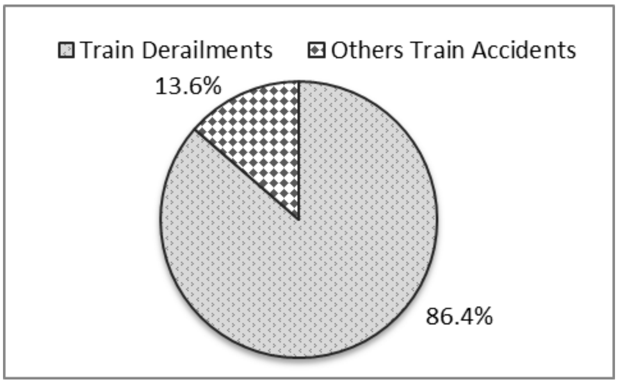
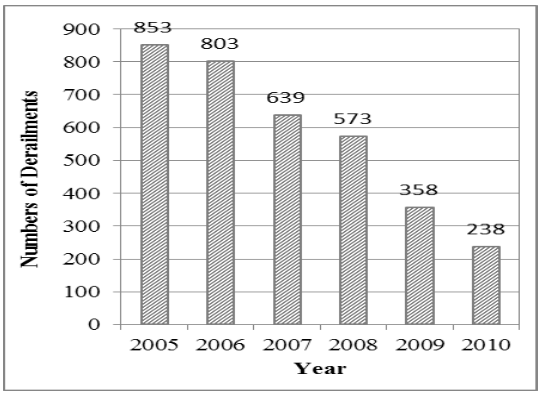


Fig. 1: Percent of Train Derailments

4.2 Trend And Rate Of Train Derailments

Year wise derailment shows a downward trend over the next years after 2005 which indicates that the situation is improving. It has been found from fig. 2 that train derailments were more than 3 times higher in 2005 than in 2010.



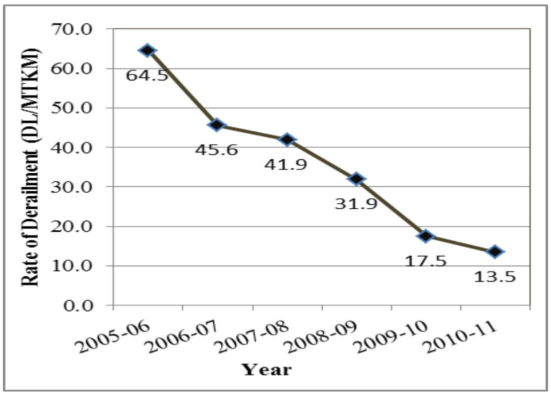


Fig. 2: Year wise trend of train derailments Fig. 3: Rate of Train Derailments (DL/MTKM)

Rate of train derailment was calculated in terms of Derailments per Million Train Kilometers (DL/MTKM) which is shown in fig. 3. Rate of derailment (DL/MTKM) was very high in 2005-06 and the value was 64.5 which reduced to a value 13.5 in 2010-11.

4.3 Train Derailments Contributing Elements

Table 1 shows yearly statistics of train derailments in Bangladesh Railway due to various elements. It has been found from analysis that among 3464 nos. of train derailments about 87.2% (3022 nos.) has been investigated and 12.8% (442 nos.) of derailments remain uninvestigated. Thus elements of train derailments established only for 3022 nos. of investigated train derailments.

Table-1:Train derailments distribution by various elements

Year	Operational Elements (A)	Infrastructure Elements (B)	Rolling-Stock Elements (C)	Combined Elements (D)	Others
2005	96	546	34	24	1
2006	73	505	42	15	1
2007	92	427	51	25	0
2008	83	368	40	36	0
2009	65	222	25	30	0
2010	58	127	22	14	0
Total	**467 (16%)**	**2195 (73%)**	**214 (7%)**	**144 (4%)**	**2**
Numbers of Derailments Investigated =					**3022 (87.2%)**
Numbers of Derailments Remain Uninvestigated =					**442 (12.8%)**
Total Numbers of Derailments =					**3464**

Analysis of train derailment by elements shows that most of these derailments took place due to infrastructure elements which was near about 73% (2195 out of 3022) and about 16% derailments occurred due to operational elements. On the other hand, 7% derailments happened due to rolling-stock elements, 4% due combined elements of total derailments. These data clearly indicates the dilapidated condition of rail tracks and inferior state of train operation in Bangladesh.

4.4 Train Derailments Distribution By Causes

Table 2 shows a various causes of train derailments from which 'top 10' causes are highlighted. It is found that the cause 'spread of gauge' was on the top of the list which shares 45% of train derailments. From this analysis it is also evident that old and worn rail, defective sleeper, inadequate ballast, insufficient fittings-fastenings etc. were found to be the potential factors for infrastructure caused derailments. Therefore, it can be said that only strengthening of the track by providing adequate fittings-fastenings can reduce train derailments to a considerable amount. However, operation related derailments resulted from improper loading-unloading, sudden speed controls, wrong settings or manipulations of points. Again, rolling-stock caused derailments aggravated by axle and wheel defects, failure of spring, bogie and suspension, defective buffer, coupling failure, equalizing beam broken etc.

Table 2:Distribution of derailments by causes

SI NO.	Elements Category	Causes of Derailments	Derailments	
			Nos.	(%)
1	Infrastructure	Spread of Gauge	1361	45.0
2	Infrastructure	Large Number of Worse Sleeper	321	10.6
3	Infrastructure	Shrinkage of Track	207	6.8
4	Infrastructure	Level Difference in Track and Tight Gauge	93	3.1
5	Infrastructure	Breaking and Wearing of Rail	56	1.9
6	Infrastructure	Superelevation Problem	16	0.5
7	Rolling-Stock	Equlising Beam Broken	17	0.6
8	Rolling-Stock	Shakle Pin Broken	21	0.7
9	Rolling-Stock	Axle and Wheel Defect	52	1.7
10	Rolling-Stock	Sudden Speed Control	165	5.5
11	Operational	Wrong Settings & Manipulations of Points	83	2.7
12	Operational	Breach of Rules, Signal Violation	50	1.7
13	Operational	Wheel Traverse in Two Routes	40	1.3
14	Operational	Interlocking Defects	11	0.4
15	Operational	Obstruction on Track	21	0.7
16	Infrastructure	Defects in Points and Crossings	45	1.5
17	Infrastructure	Track Settlement due to Heavy Rain and Loose Soil Strength	19	0.6
18	Operational	Unbalanced or Skew Loading	33	1.1
20		Miscellaneous Causes	411	13.6
		Total Numbers of Derailments =	3022	100

Source: Department of COPS, Bangladesh Railway.

4.5 Distribution Of Derailments By Track Gauge

Bangladesh Railway has three types of track gauge MG (Meter Gauge), BG (Broad Gauge) and DG (Dual Gauge) with a proportion of 64%, 23% and 13% respectively. Intensity of train runs on various gauges of track also varies considerably. Statistics shows that average train kilometers on BG track including passenger and freight train was 26% while on MG track was 74% of total train kilometers for a period of six years from 2005-10.

Table-3:Train derailment based on track gauge

Year	Nos. of Derailments		
	BG	**MG**	**Total**
2005	83	770	853
2006	96	707	803
2007	46	593	639
2008	46	527	573
2009	44	314	358
2010	35	203	238
Total (%)	**350 (10.0%)**	**3114 (90.0%)**	**3464**

Analysis of derailment based on track gauge for the above mentioned period shown in table 3 reveals that among 3464 nos. of train derailment only 10% of derailment took place on BG track while remaining 90% derailments occurred on MG track. This result reveals that MG track has more prone to derailments due to its lower stability compared to BG track.

4.6 Distribution Of Derailments By Line Types

Distribution of derailments based on line type shown in fig. 4 depicts that maximum numbers of derailments were took place on branch line (46.2%) while minimum numbers of derailments occurred on loop line (8.1%).

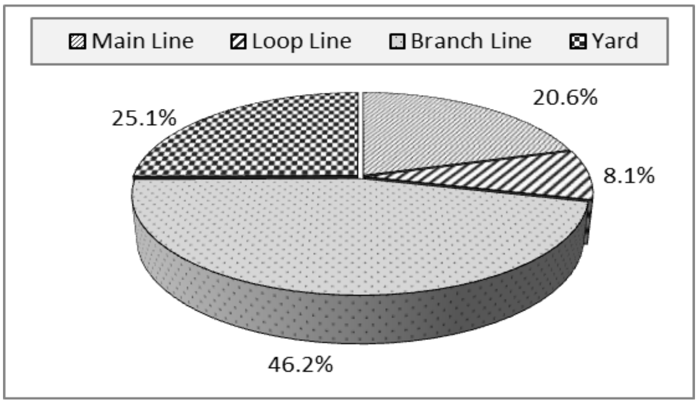


Fig. 4: Distribution of train derailments by line types

Beside these, a substantial amount of train derailments (around 21%) also occurred on main line and 25% derailments happened at yard. Most of the yard derailments were the effects of careless shunting operation. These analysis results exposes that condition of branch line were more favorable for train derailment to occur.

4.7 Distribution Of Derailments By Train Types

Table 4 shows the distribution of derailments based on train types from the period 2005-10. About 61% derailments occurred in passenger trains, 5% on freight train and remaining 34% on others train (i.e. reck, shunting, locomotive & miscellaneous). However, derailments in intercity, mail & express, container express and shuttle train were in same trend during the analysis periods. Besides, derailments on local trains and derailments on shunting were reduced to a large extent.

Table-4:Train derailment based on train types

Train Types	Train Derailments Numbers						
	Total	2005	2005	2007	2008	2009	2010
1.Passenger Train	**2107 (60.8%)**						
Intercity	116	20	21	22	21	20	12
Mail & Express	431	101	93	84	56	46	51
Local	997	247	231	195	161	91	72
Shuttle	331	100	103	74	37	9	8
Special	232	47	29	33	47	38	38
2.Freight	**173 (5.0%)**						
Freight	40	17	10	6	2	2	3
Containers Express	133	25	30	34	21	13	10
3. Others	**1184 (34.2%)**						
Reck	25	3	1	7	6	7	1
Shunting	519	162	141	95	62	41	18
Locomotive	23	8	4	5	5	0	1
Miscellaneous	545	106	128	71	134	88	18
Type Unknown	72	17	12	13	21	3	6

Source: Department of COPS, Bangladesh Railway.

4.8 Average Block Time Per Derailment

Table 5 represents the average block time [hrs:min] per train derailment in Bangladesh which indicates an average value of 7:00 [hrs:min] per derailment during the analysis period 2005-10. It is also important to say that the total block time in six years were equivalent to 1012 days or 167 days/year which indicates a serious disruption in train service.

Table-5:Average Block Time per Derailment

Year	No. of Train Derailments	Total Block Time [hrs:min]	Average Block Time [hrs:min]
2005	853	4111:20	4:49
2006	803	7034:52	8:45
2007	639	4511:15	7:03
2008	573	4299:15	7:30
2009	358	2863:59	8:00
2010	238	1449:21	6:05
Total	**3464**	**24270:02**	**7:00**
Total Block Time is Equivalent to 1012 Days			
Block Time in (Days/Year) = 167 days/year			

4.9 Derailment Prone Locations Identification And Derailment Intensity Map Of BR

Based on the frequency of derailments occurred at various sections and stations including main line, loop line, branch line and yard throughout the analysis period 2005-10 numbers of derailments are organized for Bangladesh railway to determine derailment prone sections and stations. A range of derailment frequency has chosen as per table-6 on the basis of the pattern of derailments occurrence per 6-years to classify various sections and stations into four groups such as normal, low, moderate and high derailment intensity zone. Major derailment prone locations has been listed in table 7 with various details.

Table-6:Classification of derailment zone based on derailment frequency

Derailment frequency/6-Yrs	***Zone Category***
0-1	Normal Intensity Zone
2-4	Low Intensity Zone
5-9	Moderate Intensity Zone
Over 9	High Intensity Zone

A map of Bangladesh Railway (fig. 5) is prepared showing derailment intensity based on the frequency of derailments at various sections and stations. From derailment intensity map several routes have been found with high intensity of derailments at most of its sections and stations. The major derailment prone routes are Tista-Ramna Bazar, Lalmonirhat-Burimari, Kanchon-Panchagarand Rajshahi-Amnura, Mymensingh-Mohanganj, Moglabazar-Chatak Bazar, Sholashahar-Dohajariand, and Akhaura-Laksham. On the other hand, major derailment prone sections are found as Mymensingh-Shambhuganj and Rajapur-Sadar Rasulpur for main line while Hatibandha-Votmari and Netrokona-Thakurakona sections for branch lines. Main causes of frequent derailments at these sections and routes were the insufficient strength of the track due to inadequate track components. Thus, these sections/routes need immediate attention for track rehabilitation/reconstruction to minimize derailments in future.

Table-7:List of major derailment prone locations

Line Types	**SI No.**	**Section/Station Name**	**Numbers of Derailments**				
			Total	**Main Line**	**Loop Line**	**Branch Line**	**Yard**
Main	1	**Mymensingh - Shambhuganj**	**31**	**28**		1	2
	2	Moglabazar - Sylhet	17	16			1
	3	Bahadurabad Ghat -	16	16			
	4	Rajapur-Sadar Rasulpur	22	21	1		
	5	Fajilpur-Muhuriganj	14	14			
	6	Lalmai-Mainamati	11	11			
	7	Goalanda Ghat-Goalanda	24	24			
	8	Isshardi	17	13	4		
	9	Goalanda Bazar-Pachuria	10	10			
	10	Kaunia	11	8	3		
	11	Altafnagar-Nasharatpur	4	4			
	12	Mahendranagar-Tista	4	4			
Branch	1	Netrokona - Thakurakona	60			60	
	2	Afjalabad - Khajanchigaon	53	3		50	
	3	Netrokona - Shyamganj	51	1		50	
	4	Dohajari-Kanchan Nagar	30			30	
	5	Foujdar Hat-CGPY	30			29	1
	6	Kanchan Nagar-Patia	23			23	
	7	Rajshahi-Shitlay	23			23	
	8	Amnura-Chapainababganj	16		1	15	
	9	Amnura-Kakonhat	9			9	
	10	**Hatibandha-Votmari**	**111**			**111**	
	11	Aditmari-Kakina	65			65	
	12	Kakina-Votmari	62			62	
Loop	1	Gouripur	26	10	9	3	4
	2	Shohagi	4		4		
	3	Akhaura	12	1	3	8	
	4	**Lakhsham**	**45**	**5**	**15**	1	24
	5	Fajilpur	11	2	9		

	6	Comilla	16	1	7		8
	7	Goalanda Ghat	8	2	6		
	8	Goalanda Bazar	9	4	5		
	9	Chuadanga	5	1	4		
	10	Rangpur	5	1	4		
	11	Annadanagar	4	1	3		
	12	Rangpur-Shayampur	2	1	1		
Yard	1	**CGPY, CGMY and CGD**	**414**				**414**
	2	Newmouring	132				132
	3	Chittagong	31		2		29
	4	Mymensingh	99	13	10	2	74
	5	Dhaka	37		2		35
	6	Sylhet	24		1	1	22

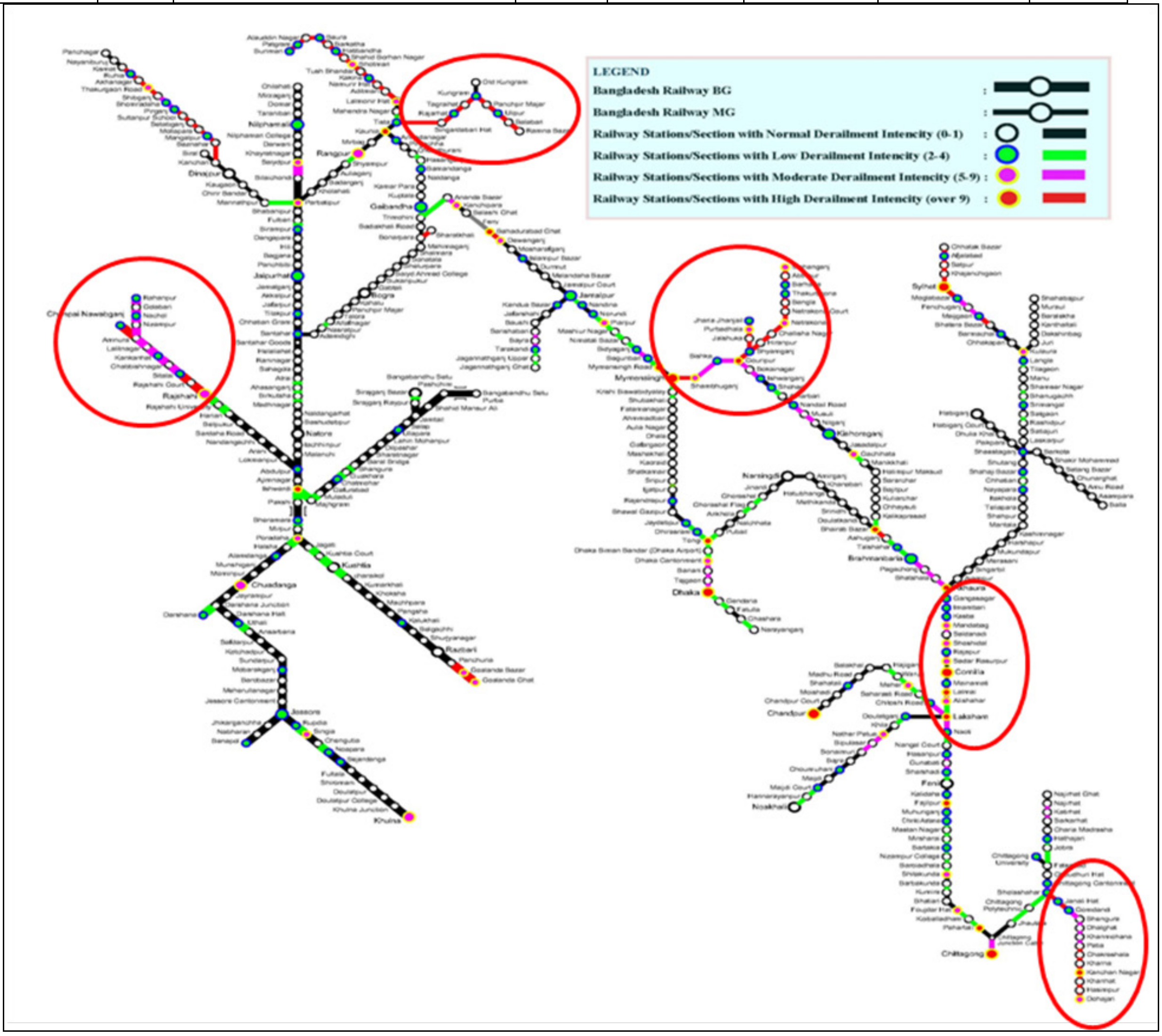


Fig. 5 : Bangladesh Railway Route Map (Showing derailment intensity at various sections and stations)

5. IMPROVEMENT OPTIONS

Observing the train derailments scenario in Bangladesh following options can be implemented to enhance the rail safety.

- Existing track condition should be assessed properly (especially for the derailment prone routes track) and renovation of rundown track should be done as early as possible specially the branch lines.

- Regular maintenance and scheduled inspection of tracks should be ensured by the authority.
- Defective sleepers as well as track having inadequate fittings-fastenings must be restored immediately.
- Points and crossings should be kept in perfect geometric and structural condition as well as operation of these points must be handled carefully.
- Conversion of MG track into BG can be done progressively with concrete sleeper instead of using wooden or steel sleeper.
- Age old or faulty vehicles like engine, loco or rolling stocks should be restricted from railway services and new engines and coaches need to be procured soon.

6. REFERENCES

Agarwal Amitabh, (2005). Rail Accidents due to Human Errors, Indian Railways Experience, International Railway Safety Conference, Cape Town, South Africa.

Agarwal M.M., (2007), 'Indian Railway Track.' Design, Construction, Maintenance & Modernization, 15th Revised & enlarged edition.

Amitabh, P. C. Sharma,(2004). "Safety as Key Business Theme! - Indian Railways Perspective'' Govt. of India, Ministry of Railways.

Arora, S.P. and Saxena, S.C., (2006), "A Text Book of Railway Engineering", 7th edition, New Delhi, Ish Kanpur.

Bangladesh Railway (BR) website, (2011).www.railway.gov.bd.

Bangladesh Railway,(2009).Bangladesh Railway Information book-2009.

Bangladesh Railway,(2010).Bangladesh Railway Information book-2010.

Barkan, C.P.L., Dick, C.T. & Anderson, R.T., (2003). Railroad Derailment Factors Affecting Hazardous Materials Transportation Risk, Transportation Research Record, No. 1825.

Blader, F.B., (1990). A Review of Literature and Methodologies in the Study of Derailments Caused by Excessive Forces at the Wheel/Rail Interface, Association of American Railroads Report R-717, December 1990.

Brabie. Dan., (2005). On the Influence of Rail Vehicle Parameters on the Derailment Process and its Consequences.

Delivering a Sustainable Transport (2007), Department for Transport, Great Minster House, 76 Marsham Street, London SW1P 4DR.

H. Wu and N. Wilson., (2006). Railway Vehicle Derailment and Prevention, pp. 209-237.

Mundrey. J.S, (2010), 'Railway Track Engineering', 4th Edition, 7 West Pate Nagar, New Delhi 110 008.

RailSafety New Zealand, (2011).Website: www.railsafety.co.nz/approach.html

Wikipedia,http://en.wikipedia.org/wiki/De-railment, 2011

Wu, H. and Elkins, J., (1999). Investigation of Wheel Flange Climb Derailment Criteria, Association of American Railroads Report R-931, July 1999.

EVALUATION OF CURRENT OFF-STREET PARKING PRACTICE IN DHAKA CITY

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ABSTRACT

This paper attempts to focus on the current practice of off-street parking facility in Dhaka city. With a view to reduce congestion by solving parking problem, several off-street parking facilities have been constructed in Motijheel commercial area. Field investigation suggests that the facilities are constructed without carrying out Traffic Impact Assessment (TIA). Most importantly, these off-street facilities are also serving as commercial complex in addition to parking space which will in turn trigger parking generation. Moreover, these facilities provide options only for cars whereas here only 5.1% trips are made by cars while 40-60% by para-transit. That is why, instead of solving problem rather it is expected that it would create more traffic problem. To see the adverse effects of parking-cum-commercial space, study was conducted in two off-street parking facilities in Motijheel commercial area namely Sadharon Bima Bhaban and Motijheel City Centre. It was found that out of 385 parking lots, 315 were allotted for commercial spaces located within the structure while only 70 were rest for outside parkers in Sadharon Bima Bhaban which forced the drivers to park on-street. Finally, the study discourages capital intensive supply based solution provided by Dhaka City Corporation (DCC) and RAJUK (Rajdhani Unnayan Kartipakkha).

Keywords: *Off-street parking, TIA, commercial space, para-transit.*

1. INTRODUCTION

In recent times with rapid growth of population , Dhaka city have seen a large increase in road traffic and transport demand, which has consequently lead to deterioration in capacity and inefficient performance of traffic systems. A recent study by Roads and Highways Department (RHD) has estimated that, the traffic congestion in Dhaka causes a loss of Taka 19,555cr a year (The Daily Star, 2010). Several steps are taken for the smooth transportation system. But it should be also bear in mind that a car runs only 400 hours of a year leaving 8360 hours being stationary. So, there should be also concern about these 95% of time being idle (Kadiyali, 2004). There are lots of talking about moving vehicles but the congestion problem is also attributed by stationary ones which take a significant portion of a road. With the growing of car ownership day by day and side by side lack of adequate parking facilities, the problem of parking has become a serious issue. In Dhaka, up to 2003 number of registered private cars were 87,866 whereas after October, 2013 it climbed up to a grand total 1,90,515 (BRTA, 2013) which indicates more than doubling the figure in 10 years only. The total number of motor vehicles is 7,70,895 after October, 2013 showing 24.7% of private cars which caters only 5.1% of total trips. (Development of EST in Bangladesh, 2010). Obviously, these increasing private cars require not only street space to move but also require space to park as well as loading – unloading space. Now, considering the fact that 7,944 auto-rickshaws and almost 6,00,000 rickshaws are carrying almost 45% of trips which is much higher than that of car- logically emphasis must be given for their temporary parking needs. But, unfortunately, the authorities viz. Rajdhani Unnayan Kartripakkha (RAJUK) and Dhaka City Corporation (DCC) are concerned only about car parking even though unlike western countries, the cars in Bangladesh also need loading-unloading facility in front of any commercial establishment as the cars are not self-driven rather chauffeur driven which essentially suggests loading-unloading facility should be given more emphasis than basement parking. Again, city authorities are seeking supply based solution by constructing off-street parking structures like western countries. Barter (2011) commented on his report that this city is promoting parking supply like automobile dependent western cities which will fuel the growth of car ownership and traffic. Additionally, these parking structures are also serving as commercial space which is in-turn triggering traffic. In Motijheel, there are two off-street parking structures namely Sadharon Bima Bhaban and Motijheel City Centre. Both are serving as parking-cum-commercial space. In the study, it was found that, major portion of the parking space belongs to the offices located in the building while other users find it difficult to have space. To see the adverse effects of ignoring above mentioned aspects, study was conducted in Sadharon Bima Bhaban and Motijheel City Centre which is presented in the following sections.

2. METHODOLOGY

The study was a behavioural science rather mathematical. Some data was extracted from the off-street parking structure's working drawing viz. parking capacity. Based upon the interviews of the building officials, additional information was collected. Parking inventory of the off-street parking structures was done by filed investigations. Then, the building code guidelines about parking were compared with the filed investigation. Throughout the study, photogrammetric survey provided useful information.

3. FINDINGS

Findings on two off-street parking structures are presented in the following sections.

3.1 Sadharon Bima Bhaban

Sadharon Bima Corporation has initiated the country's first multi-storied off- street parking feature in Dilkusha, Dhaka back in year 2006. It has the capacity of holding 385 vehicles at a time. It has a foundation of 20 storey building with 3 basements. Currently it is 9 storied and 11 more stories are yet to be constructed. Each floor is approximately 20973.93 square feet.

Apparently it seems good to have a multi storied parking but all of the car users are not getting benefits from the structure because of commissioned parking. It is reserved parking for few offices. The basement parking is fully dedicated to the commissioned parking.

Table 1: Allotment of Basement Parking

Basement	Reservation	Number of reserved parking
Basement-1	Bima Corporation	40
Basement-2	Rupali Bank	40
Basement-3	Fisheries Corporation	40

So, it can be seen, already 385-120=265 is available for the outside parkers. The biggest objection is, the reserved space is not utilized full. It has booked for 40 car spaces but 5-6 cars are present at a time. The cars have a sticker so that the parking officials recognize that this car is for reserved space.

Going to the superstructure, it has been seen that the same scenario exists for reserved parking. AB Bank, Popular Life Insurance, Rupali Insurance, Prime Bank etc have reserved parking. The reserved parking is 175 in upper stories- an official informed. Some allotments are given below.

Table 2: Some allotments of some reserved parking in superstructure

Organization	Number of Reserved Parking
Prime Bank	30
Popular Insurance	10
Rupali Bank	20

So, now available space for outside parkers is now 265-175=90. The 4 stories above the parking have already been allotted for commercial space. Rupali Bank, Rupali Insurance, Prime Bank has already their offices there. In future, 11 more stories will be constructed. Total floor spaces for 11 stories=11*20973.93=230713.23 sft. According to Bangladesh National Building Code (BNBC, 1993), Part-3, Chapter-1, Section-1.9- Commercial space (i.e. Office) should have 1 parking space per 200 sq.m or 2150 sq.ft almost. If all those stories are constructed than according to the code required number of car space would be =(20973.93*11)/2150=107.3, almost 110 spaces. So, no space is available for the outside parkers. So, the employers in the offices inside the building will find it difficult to park their own vehicles.

Figure 1: Reserved parking & its consequence on roads.

So, Figure 1 justifies the previous page's description. Empty spaces in off- street facility but parked vehicle on road just in front of the off-street parking structure.

3.2 Motijheel City Centre

Motijheel city centre is the tallest building with 37 stories of the country situated in Motijheel commercial area having a total floor area of 482, 413 square feet (Orion Group, 2012). The main objective of building such massive structure was to alleviate the parking problem of the area. Accordingly, 10 parking floors are dedicated but unfortunately it has become a problem generator rather than solving previous ones.

This giant structure is situated just in the mouth of a T- junction. Moreover, it is situated beside a busy road without keeping any offset from the main road. The position of the structure is shown in the following figure.

Figure 2: Position of City centre just in mouth of a T-junction.

Actually, this building is parking-cum-commercial complex. 27 stories here are for commercial complex. Additionally, it will host gymnasium, convention centre, restaurants, banks etc. So, these will again attract trips for which there will be more parking demand which is not considered during planning.

Again, no TIA was done prior to this mega development. Whether this development will have any negative effects on active roads or not was completely neglected. Even this large development has no offset from the main road. So, before going into full operation, the building itself is creating chaos in the area, so it is easily understood what would happen when the building will be in full operation. A thorough calculation is done here. The parking capacity of the building is given in Table-3.

Table 3: Parking Capacity of Motijheel City Centre

Floor	Capacity
Basement 1,2	47 each
1st to 6th parking level	48 each
7th parking level	26

This is leading to a total of (47*2) + (48*6) + 26= 408 parking lots. Now, assuming that the building has uniform floor area in each floor which states (482,413/37)= 13000 sft almost in each floor. So, 30 stories will have (30*13000) = 390,000 sft gross area equivalent to 36,229 sqm area. So, according to BNBC (Part-3, Chapter-1), it requires (36,229/200) = 181 parking space. But it provides 408 spaces. So, only (408-181) = 227 spaces whereas the demand of parking in the road is much more than it. Ahmed (2013) showed that, the demand of parking just in the front road of this mega structure is found almost 600 vehicle-hour. In a study of Dhaka Transportation Coordination Authority (DTCA), the parking demand is to be found around 4000 vehicles/day in the Motijheel commercial area (Topbdnews, 2012). Again, it can be told from Sadharon Bima Corporation off-street parking structure that, those 227 spaces would definitely be occupied by the offices located in the city centre. So, ultimately on-street parking would prevail.

The following figure would more clear the above discussion.

Figure 3:T-junction in front of Motijheel City Centre.

This is the T-junction just in front of the structure. Prior to the full operation of the building, the corridor is suffering from congestion due to on-street parking. When city centre will be in full operation, there will be more on-street parking owing to trip generated by the structure.

3.3 Other Supply Structures

As it has been said that the authority is keen to circulate around supply oriented solution, their recent activities have proven it again. After DCC, it is now RAJUK who is implementing off-street parking features. If it was only for parking features only, then significant benefits might be possible but all these are parking-cum-commercial complex.

RAJUK has inaugurated a 14-storey commercial building in the capital's Gulshan which offers a 228-vehicle parking facility in its two basements and first seven floors. Again, there is another multistoried car parking to be built in Motijheel in RAJUK's own land. Its capacity will be 285 at a time (Deshsomoy, 2010). Here prevails the previous scenario, it is also a parking-cum-commercial complex. So, authorities are reluctant on strong management and strict enforcement of laws rather on capital intensive work.

4. CONCLUSIONS

Current off-street parking structures cannot alleviate the current parking problem rather it is acting as a parking generator. As these parking structures are also acting as commercial spaces, it will further invite more traffic and the parking spaces will be occupied by the vehicles of those commercial spaces. Again, as most of our trips are carried by other than cars, so transitory parking (loading- unloading space) provision should be kept in front of every commercial building.

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5. REFERENCES

Ahmed A., Binoy T.H., Abeer M. M., "Evaluation of Motijheel City Centre Off-Street Parking Facility" (2013), International Journal of Science and Engineering Investigations, Vol-2, Issue 19, pp. 85-88.

Bangladesh National Building Code, 1993, Part-3, Chapter-1, Page: 3-13.

Barter P. (2011), "Parking Policy in Asian Cities", (Consultant's report), Financed by ADB under RETA 6416: A Development Framework for Sustainable Urban Transport – Parking Policy in Asia.

Bangladesh Road Transport Authority (BRTA)- Number of registered vehicles in Dhaka city. Available at http://www.brta.gov.bd/index.php/statistics accessed on 04 December, 2013.

Development of Environmentally Sustainable Transport (EST) in Bangladesh, 2010. Available at http://www.uncrd.or.jp/env/5th-regional-est-forum/doc/04_Breakout%20Session%201%20(Country)/BG2/BS1-2-1_Bangladesh.pdf accessed on 04 December, 2013.

Kadiyali, L.R. (2004), *Traffic Engineering and Transport Planning*, Seventh Edition, p-287.

Orion Group (2012), Official website of Orion group available at http://www.orion-group.net/city_center.php, Accessed on 04 December, 2013.

The Daily Star (2010), Online version available at http://www.thedailystar.net/newDesign/news-details.php?nid=147756, Accessed on 04 December, 2013.

The Topbdnews (2012), Online version available at http://www.topbdnews.com/index.php?option=com_content&view=article&id=3583:2012-01-07-05-12-20&catid=36:hitec&Itemid=27 accessed on 20 January, 2013

The Weekly Deshsomoy (2010), Online version available at http://www.deshshomoy.com/index.php?option=com_content&view=article&id=982&Itemid=50&cal_offset=0n Accessed on 04 December, 2013.

INSTITUTIONAL WEAKNESS AND CONSEQUENTIAL IMPACT ON LAND USE AND TRANSPORTATION SYSTEM IN DHAKA METROPOLITAN CITY

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ABSTRACT

The proper institutional and organizational arrangements are required to handle and administer the functions and performance of transportation system. From 1906 to 1991, Dhaka's area grew by 58 fold and its population by more than 35 fold. Many organizations were formed to deal with this tremendous growth. There are about 51 organizations under four ministries related directly (viz. DCC, RAJUK, DMP) or indirectly (viz. BR, RHD, DESA, WASA) with transportation system of Dhaka City. Inadequate and inappropriate policies, lack of a vision, overlapping jurisdiction of different agencies/ departments/ institutions, absence of public and private participation, undefined role of private sectors are also the major institutional weakness of the city which are deteriorating the existing land use and transportation situations as well as overall condition of the city. This paper is particularly discussed the organizational/institutional weaknesses which are directly or indirectly related to transportation system with particular emphasis on policy and functional issues and their consequence impact on land use and the transportation system in Dhaka Metropolitan City. Finally, a set of recommendations have been put forward to improve the performances and effectiveness of the existing organizations/institutions as well as overall transport system of the city.

Keywords: *Institution, Impact, Transportation, Dhaka City*

1. INTRODUCTION

It has been recognized by development agencies for a long time that the present institutional framework and the capacities of institutions in the urban sector are major constraints to urban development management in Bangladesh. The institutional and organizational arrangements are required to handle and administer the functions and performance of transportation system. From 1901 to 2001, Dhaka's area grew by 30 fold and its population by over 100 fold (Mahmud, 2009). Many service delivery organizations were created to deal with this tremendous growth.

The principal organization responsible for city management and welfare of its citizens is DCC. A crisis situation persists pertaining to the governance of Dhaka City and the one dimension of the crisis is the inherent weakness of the Dhaka City Corporation (DCC) itself due to its institutional and management deficiencies, personnel and capacity deficiencies, and resource constraints. Although an elected body, DCC is not sufficiently transparent and accountable to the people. Rajdhani Unnayan Katripakkha (Capital Development Authority or RAJUK) provides urban housing and city planning is independent of DCC and the coordination between them is ineffective. It is a self-financing organization always tries to accomplish all of the space, as a plot to increase the value of plots without considering frontage and access road or functionality of the existing road, even at junction point where corner widening is must, that space sells as plot too. Indeed, now it becomes a revenue generator instated of city developer which is mandated for the faulty development policy of the organization. The Water and Sewerage Authority was created to provide drinking water and drainage, the Dhaka Electrical Supply Authority to provide electricity. These organizations are independent of Dhaka City Corporation (DCC) and the coordination between them is ineffective. Indeed, there is a very little integration and co-ordination between the different organizations, institutions of the city resulting frequent cutting and digging of the road, water logging, haphazard development of unbuildable and marshy land, conversion of residential area, unplanned growth of fringe areas etc.

This paper identifies a bulk of system deficiencies of organizations concerning transport sector of the city with particular emphasis on policy and functional issues and their consequence impact on land use and the transportation system in Dhaka Metropolitan City. Data has been collected particularly from the secondary sources like different organizations, previous reports etc.. In addition, discussion with the different concerned authorities and opinion survey was made to collected information regarding organization setup, responsibilities and associates problems etc. Finally, some policy and concerned institutional measures are recommended in the paper.

2. CONCERN ORGANIZATIONS

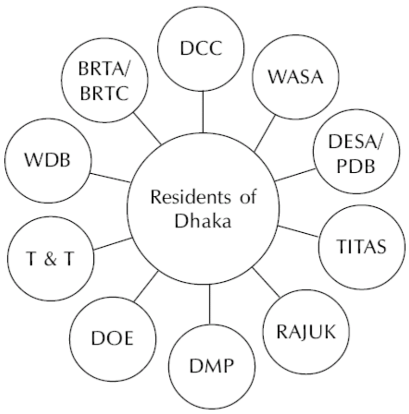
There are about 51 organizations directly or indirectly related with transportation system of Dhaka City. Among them, some of the organizations are directly involved with the land use and transport system development and some have not direct involvement with the system but make impact on the overall system like wasa, T&T etc. The major organizations which are directly concern with the land use and transport system are:

1. Dhaka City Corporation (DCC)
2. Rajdhani Unnayan Kartripakka (RAJUK)
3. Bangladesh Road Transport Authority (BRTA)
4. Traffic Division, Dhaka Metropolitan Police (DMP)
5. Dhaka Water and Sewerage Authority (DWASA)
6. Bangladesh Road Transport Corporation (BRTC)
7. Dhaka Transport Coordination Board (DTCB) recently renamed as Dhaka Transport Coordination Authority (DTCA)

Many other organizations also have influence on the city's transportation system, which can be enlisted as follows:

1. Roads arid Highways Departments (RHD)
2. Bangladesh Railway (BR)
3. Dhaka Electric Supply Authority (DESA)
4. Titas Gas
5. Telephone & Telegraph (T & T) etc.

Some major organizational setup of Dhaka city and their services are listed in Figure1 below:



Acronym	Organization	Service
DCC	Dhaka City Corporation	Municipal service
WASA	Water and Sewerage Authority	Pure drinking water Sewerage drainage
DESA/PDB	Dhaka Electric Supply Authority Power Development Board	Supply of electricity
TITAS	Titas Gas Transmission and Distribution Company	Gas supply
RAJUK	Rajdhani Unnayan Kartripakyha	Development plan
DMP	Dhaka Metropolitan Police	Law and Order
DOE	Department of Environment	Environment
T & T	Telegraph and Telephone Board	Telephone service
WDB	Water Development Board	Flood control
BRTA/BRTC	Bangladesh Road Transport Authority/ Bangladesh Road Transport Corp.	Transportation

Figure 1: Major Organizational Setup of Dhaka City

Four following Ministries are intensely related with the transport affairs:

1. Communication,
2. Public Works,
3. Local Government and Rural Development,
4. Home Affairs.

3. ORGANIZATIONAL RESPONSIBILITIES AND CONSTRAINTS

3.1 Dhaka City Corporation (DCC)

The first civic committee created to consider solutions to urban problems in Dhaka was the Committee of Improvement, formed in 1823. It was reshaped as the Dhaka Municipal Committee in 1864 and entrusted with various public works of civic amenities. The concept of administering the municipality through elected representatives was introduced in 1884 and the Chairman, Vice-Chairman, and two thirds of the commissioners were elected directly by the people. The municipality was given the status of a corporation in 1978 (Rahman, 2004), and in 1990 it was renamed the Dhaka City Corporation and was divided into 10 zones to fulfill the objectives of decentralization. In 1993, the government with a view to democratizing the city corporation, made drastic amendment to the 1983 ordinance and provided that the mayor and the commissioners would be elected by direct election on adult franchise. According to the law, the executive powers of the corporation is vested in and exercised by the mayor and the term of the elected body is five years. The corporation constitutes eight standing committees and other committees to monitor and guide the diversified activities of the organization. The Chief Executive Officer, who is assisted by the Secretary, the Heads of Departments and Zonal Executive Officers, assists the Mayor. About 11,000 employees carry out various duties, catering to civic need (DCC, 2004).

3.1.1 Organizational Setup of DCC

DCC has a huge staff, numbering more than 10,000 (Table 1). From this Table it is seen that class I staff is almost double than the class II and class III staff is almost 4.5 times higher than the class IV staff. This obviously breaks the pyramid of staffing pattern and creates an imbalance structure of staffing pattern. Almost 60 percent class III staffs are temporary results unaccountability and imbalance in the working responsibility (Mahmud, 2009).

3.1.2 Problems and Deficiencies of DCC

The principal organization responsible for city management and welfare of its citizens is DCC. This organization lacks vision commensurate to the size and character of Dhaka. The people in this organization do not appreciate the complexities of a city system. DCC is weak administratively, managerially, and financially. It is also inadequately staffed. Although an elected body, DCC is not sufficiently transparent and accountable to the people.

The one dimension of the crisis is the inherent weakness of the Dhaka City Corporation (DCC) itself due to its institutional and management deficiencies, personnel and capacity deficiencies, and resource constraints. The central government agencies control utility supply, sewerage, environment, traffic, law and order, city planning and development, and public housing. Again, there is a serious lack of co-ordination between the city authority and the various governmental agencies. A short description of the major weakness is described in below:

Lack of Coordination between DCC and WASA in the Operation and Maintenance of the Sewer Lines and Storm Water Drains: Most of the Engineers of DCC are involved transport activities in the sense that most of them are connected with the construction and maintenance of roads. There is a lack of coordination between DCC and WASA in the operation and maintenance of the sewer lines and storm water drains of WASA. Due to blockages of these storm water drains, there is overflow of polluted water causing inconvenience to the city dwellers. In most cases, people lodge complaints to the DCC even though this is not the responsibility of DCC.

Conflict between RAJUK and DCC: In "Uttra Model Town" area of DCC, where residential and commercial plots were allotted to different persons by RAJUK without making provision of sewerage lines and storm water drains yet. Surface development with road network plan conducted by RAJUK and they sell the plot. Most of cases like Uttra, they did not develop subsurface through providing utility lines like sewerage, gas, storm water drainage line etc. After the completion of the surface development and plot allotment under RAJUK, the developed area is shifted to the DCC for maintenance. As RAJUK could not required to provide any completion report to DCC, when DCC go to the maintain this as well as to go to collect the revenue from the land owner, they face several problems for the incompleteness of the necessary facilities which was the responsibility of the RAJUK. At that time it is also becomes very difficult to provide subsurface utility facility as most of the surface areas already being developed by constructing road or building. The land from where RAJUK is developing and earning money, DCC is facing local problem for the fault of RAJUK in that place and loosing income for the providing of additional facility which should provided the developer, RAJUK at the preliminary stage. This is one of the major causes for the conflict between the DCC and RAJUK and they always try to act independently.

Lack of Coordination and Integration between DCC and DMP: On the basis of the recommendations of the Dhaka Integrated Transportation Study (DITS), a Traffic Engineering Department (TED) was created in DCC. This department has 18 staff comprising Engineers and support staff. The department is supposed to control and manage the traffic of Dhaka city, but in fact they have very little to do with the management of Dhaka's traffic. This has happened due to the fact the engineers and the staffs of TED are not adequate trained and they have a limited knowledge of the science of traffic control and traffic engineering. Because DCC does not undertake this role, the DMP plays the main role to fill up the vacuum. DMP undertake these traffic duties their staff is overstretched but cannot perform enforcement duties effectively.

Lack of Coordination between DCC and other Organizations: Enormous problems exist in coordination and interaction among DCC and other governmental stakeholders. An effort to solve the problems of coordination was made by forming a Coordination Committee. The Minister for Local Government and the Mayor convened the Committee and the membership included the heads of 16 development and service delivery agencies. Despite all these luminaries, the Committee has failed to achieve the desired results. Since it was initiated in October 1996, it has made only marginal impact (Islam, 2001a). Meanwhile, the Minister for Housing and Public Works has also formed an Advisory Committee to advise the ministries about city planning, building codes, and architecture. This committee is composed of engineers, architects, city planners, an artist, and a historian.

Neither the Coordination Committee nor the Highways and Public Works Ministry's Advisory Committee has a single representative from the private sector, an NGO, or other component of civil society. In addition, there is also no high level of coordination between agencies at central government or metropolitan level, or indeed between private and public sector agencies. Major urban functions are divided between two ministries, neither possessing lead powers for urban development. Each is charged with the responsibility for important functions. While the Planning Commission is responsible for allocating government expenditure, it does not coordinate on a level below the national level. As a consequence, there is no strategic policy leadership at metropolitan level.

Wrong Policy Framework: As like RAJUK, as a self financing organization, DCC is more interested to construct market for generating more revenue rather than providing sufficient space for parking, setback, service road space, corner widening space etc.

Until the construction of Yousuff Market in 1913, there was no municipal market in Dhaka. Now there are big and small size 105 nos. markets of Dhaka City Corporation in the different part of the city (DCC, 2004). Every year, DCC develops and constructs many new shopping complexes in a mushrooming way occupying most of the open spaces without considering any impact on transportation even the facilities of the consumer.

For the causes of escalating development of road side market, shopping complex without considering transport impact, many of the roads of Dhaka city loses it functional capacity even become worthless. Besides this, for the lack of appropriate policy, there is a cold war between DCC and RAJUK. DCC doesn't take permission from RAJUK to construct any infrastructure. In their area, they make plan, develop by themselves without any permission or control by RAJUK or other organization. For the lack of controlled, DCC are randomly developing market in the city areas occupying lucrative junction corner areas, parking areas, even road side loading and unloading space. However, this is one of the major inherent weaknesses of the city which is come from the fundamental formulation policy problem of the city corporation.

Lack of Transparency: There is no transparency in DCC activities. The taxpayers did not often know what was being done with their money. They had no say about the work being done in their area, and more often than not nothing happened when they complained about poor quality of work. Consequent dissatisfaction among taxpayers discouraged prompt payment of municipal taxes.

Weaken Ward Commissioner: DCC presently has 90 directly elected and 18 indirectly elected ward commissioners (DCC, 2004). Energetic ward commissioners can play an effective role in furthering the interest of their wards. But it is examined that there has huge impediments like: existing facilities for orienting and training the newly elected ward commissioners are inadequate and the ward commissioners are exposed only to very limited and fragmented pieces of knowledge. The ward commissioners also lack an established office, adequate staff, and a budget allocation.

Lack of Management Information System (MIS): The lack of computerized MIS information severely hinders the smooth and speedy operation of DCC. At present DCC has a very small computer cell composed of one programmer and one computer operator. Four or five other computer operators are appointed on an ad hoc basis. In very recent, DCC has taken a project to prepare a GIS base database in each ward but unfortunately the progress of the project is not in satisfactory level (DCC, 2004).

Lacking in Decentralization of Power: DCC has also lacking in decentralization of power and responsibility as well as in transparency in its activities. The intention with which the 10 DCC zones were created and powers given to the zonal officers has not been realized. Meanwhile, the lack of transparency in the decision making and service delivery processes has resulted in a situation where citizens are unwilling to pay taxes and user charges.

The concentration at the center became so acute that citizens could not even get an application form for obtaining a trade license from the zonal office. Instead, they had to go to the DCC head office. For people spread throughout 360 square kilometers, this was an inexcusable annoyance.

Lack of Public Participatory Approach: In the present top-down approach, transport policy is developed by the responsible government agency with very little or no cognisance of other stakeholders views. Broad participation of different interest groups and consumers is essential for the effectiveness of such planning which is absent in the present practice. The developers and city planner do not usually favor public participation and the city planning preparing without proper and adequate public participation results failure of planning and programs.

3.2 Rajdhani Unnayan Kartripakka (RAJUK)

The only agency with clear responsibilities for planning and management at the metropolitan scale is RAJUK. It has occupied this central position since its creation as Dhaka Improvement Trust (DIT) in 1956, by virtue of its powers of master planning, development and building control, and development through land/estate development, area improvement and major road improvement. The jurisdiction of RAJUK comprises an area of 590 sq. miles (1,550 sq. kms), which includes the whole of Dhaka City Corporation, Narayanganj, Tongi, Savar and Joydebpur Pourashavaas and also Keraniganj, Rupganj and Siddihirgn upazilas (STP, 2004).

The major functions of RAJUK are shown below:

- Preparation of Master Plan/Development Plan for Dhaka Metropolitan Area.
- Land Use Planning and Zoning Control.
- Detailed Area Planning.
- Planning and construction of new major roads, link roads, bridges and culverts.
- Planned housing areas within the city.
- Satellite Town Development.
- Approval of markets and shopping centers.
- Implementation of special projects (like NAM Apartment Project, Prime Minister's Priority Projects).

3.2.1 Organizational Setup

RAJUK is headed by the chairman, who is assisted by five full time Members of the Board. It has 4 Divisions–Administration and Lands, Development, Finance, Estates and Planning.

In 1956, when Dhaka Improvement Trust (DIT) was established, its area of operation was only 320sq. miles (840 sq. kms). At that time, they had post to undertake their mandated duties. In 1984 as par of re-structuring of all Ministries, Divisions, Departments and Autonomous bodies undertaken by the Government the total strength of DIT was reduced to 728. At the same time the expansion of Dhaka are continued and RAJUK was required to look after and develop a larger area. Sometime later six posts were created under the revenue budget and another 55 posts were transferred from project status to the revenue budget bringing the posts to 789. Out of these 789 posts, 276 posts (about 35%) are lying vacant with a resulting loss in work output (Rahman, 2004).

3.2.2 Problems or Weakness of RAJUK

Policy Problem: It should be noted, however, that in the original legislation DIT was only a development authority: the planning powers were added by an amendment in 1958, but without apparently any organizational and staffing adjustment to match. While still institutionally and legally central, RAJUK has not succeeded in maintaining the function of strategic planning over the period since the 1959 Master Plan, and its primary function has been that of a land development and development control agency (DITS, 1994). Furthermore the focus of its activities in the land development role has mainly been on the profitable upper income sector and it has barely attempted to meet the needs of the lower income sector, which was at least a significant part of its terms of reference under the Act.

A further factor is that RAJUK has considerable effective freedom since it is only dependent on government funding for its major road improvement projects and not for its estate developments which are self-financing. The reality is that RAJUK is now regarded as one sector development agency among others. Other agencies take initiatives without reference to RAJUK equally RAJUK takes initiatives without reference to them.

Resource Problem: RAJUK is self-financed organization. It generates its financial resources from:

- The sale proceeds of developed plots (residential, commercial and industrial);
- Fees for granting approval to building plans;
- Service charges from the owners and users o plots.

Assessment of the Effect of Self Financing Authority: RAJUK is a self-financed organization and it generates its financial resources mainly from the sale proceeds of developed plots results its primary functions are now new land development and planning (mainly for the upper income sector, representing approximately the top 3.8% of Dhaka' population) and plot selling. To generate money it always tries to accomplish all of the space as a plot to increase the value of plots without considering frontage and access road or functionality of the existing road. Even at junction point where corner widening is must, sells as plot too. Indeed, gradually it becomes a real estate company under the shadow of government. By taking various advantages from the administration, it creates an unequal competition in plot sale market. Profit built-up should not major task of government public welfare organization. If it makes competition with the non-governmental organization, then as a main steersman of environmental control of the city, the necessity of this organization less down and open the various ways of corruption.

Due to the extreme shortage of staff and lack of funds (around 35% vacant of the existing post), RAJUK could not implement the Master Plan. As a result, planned development of new areas of Dhaka has not been effective. However, this type of organization is not suitable for mega city like Dhaka. If it earns money then it becomes a property developer not authority (Mahmud, 2009).

Lack of Permanent Qualified Manpower: Planning is a field of specialization. It is a very dynamic, integrated and complicated matter, in particular urban planning. There are always interacting each other. One implementation could affect all of the system. There is very little chance to overcome any fault. Transport and land use planning should have full integration. Different mode of transport should be integrated, coordinated and supportive to each other. All of these demand a group of permanent experts who have long term experience, are able to project the problem, capable to prepare long term plan and finally can implement and own the plan. Unfortunately, RAJUK is huge lack of such qualified and permanent manpower. At present RAJUK has only a skeletal planning department, with no more than a half dozen planners for its millions of people.

It is argued that city planning is a long-term task. There should permanent, responsible and highly capable and huge experienced with a visionary group of professionals who will prepare, maintain, follow-up the total city development and who would be responsible and accountable for any disorder. Indeed, most of the top level person in RAJUK comes in deputation for a certain period from different organizations. The posts of the Chairman, the Directors and most of the top-level position are filled up by people on deputation from other organizations. Even, most of the class I post is filled by deputation who come from different departments for a short time (most of the case less than 2 years). It is seen that 37 secretaries have been changed within the 53 years period (1956 to 2009) in RAJUK (Field survey, 2009). Almost two third of them tenure were less than six months. This situation is not only creating misgivings among Rajuk's officials but also negates the sense of ownership and commitment to RAJUK by the outsiders. This also not only reduces the capacity and productivity of the authority but also the responsibility as well as accountability. For the deputation recruitment for a certain period, they are not interested to take long time project. Even they are not capable to take care long term such complicated urban project as they are interchanging from one to another after a certain period. That is why, the city is developing without long term plan, and one is conflicting another.

Lack of Integration with DCC and DTCB: In order to ensure planed and coordination development of the city, land use and transport planning must be integrated. But, at present, transport planning is being shared by many organizations including RAJUK, DCC, and DTCB. As a result urban planning and development, management in Dhaka has become fragmented and uncoordinated. Consequently the integration of transport and land use plans cannot be ensured. The expansion of the road network could not keep pace with the population growth and transport demand. At the same time, physical infrastructure is being developed in Dhaka without much regard to the transport facilities to serve it. Whereas transportation land use planning should go side by side, it is seen that transportation facilities lag far behind the physical infrastructure development.

Wrong Policy in the Development Works: It has been found that the revenues received by RAJUK from the proceeds of the sales of developed plots, together with fees and charges for rendering services to the clients are inadequate to invest in the development of new roads required to connect the newly developed areas with the city road network. RAJUK prefers to build residential and commercial developments, which give quick returns or the investment. This is an act, which creates transportation problems for the newly developed areas (Mahmud, 2009).

From the field observation couple with discussion with city planner and designer it is observed that over 90 percent buildings in the capital city have been built violating the "Building Construction Rules. It is found that almost one-third land of the DMA is unbuildable marshy land due to low topography and most of the development is now progressing on that land by land filling particularly by the individual owner like eastern fringed area. Unfortunately, RAJUK never go to submerged or watered land and they do not provide any plan on that land results uncontrolled, haphazard, unplanned development of that land area like Badda, Mugda, Basabo etc and it still progressing in all around the city area. So there should have separate policy and controlling mechanism to control the haphazard development of the low land.

RAJUK provide permission for the construction of building before the construction. But after the completion, they have not required to submit any completion report. This gap influence the owner to break the permissible limit and most of the time they (almost 90 percent) violet the rules and regulation. Mandatory submission of completion report like permission report could prevent such violation.

Disproportional Capital Investment and Revenue Expenditure: The authority is required to invest its surplus funds for further investment. To accomplish this objective they must make capital expenditure on viable projects meaning that RAJUK is mostly involved in site (land development projects). As a result, they do not constructing any new roads, it is ignoring the acute parking problems being created from its indiscriminate approval of high-rise building complexes without talking into consideration their impact on the low of traffic on nearby roads. For example, in a recent survey undertaken under STP, a total of 66 tall buildings have been constructed with the approval of FAJUK which do not have any parking facilities (STP, 2004). As a result traffic congestion is a regular occurrence near those buildings. Compared with the revenue expenditure, the capital expenditures of RAUK for the development of the city of Dhaka made were very meager.

Technical Problem: RAJUK has neither any database nor on has it established a Miscellaneous Issues (MIS) in the organization. It should remove these deficiencies as a priority in order to bring dynamisms into the discharge of its mandated functions. It has been alleged that both internal and external interferences frequently create obstructions in the proper discharge of mandated functions by RAJUK. The government should address this problem seriously in order to enhance the image of the organization.

3.3 Dhaka Metropolitan Police (DMP)

Dhaka Metropolitan Police (DMP) was set up in the year 1976 through Ordinance No. 3 of 1976. With the gradual expansion o Dhaka city area, caused not only by janitorial growth but also from in-migration of people from all over the country, maintenance of law and order and traffic control became extremely difficult for the police authority. The need for setting up a separate organization for the city area was few leading to the creation of DMP.

The main functions of DMP are stated below:

- Control of crimes and maintenances of law and order in the city area.
- Control of traffic movement in the city.
- Enforcement of traffic rules in the city area to ensure road safety.
- Investigating road accidents and storing them in the micro-computer accident package (MAAP) followed by the analysis of the accident data.

3.3.1 Constraints or Problems of DMP

Major constrains which have been evaluated through the discussion with the police authority and review of the recent comprehensive study reports is described below:

Shortage of Traffic Police: Traffic police are currently assigned to approximately 232 "bits" (Point duty locations) in Dhaka. The widespread lack of discipline that characterizes Dhaka road users (both drivers and pedestrians) requires the presence of police officers at all busy intersections. Police have subsequently estimated the recommended number of bits required for effective control of traffic management is 929. At present traffic Police are stationed at only 30% of these recommended locations. To cover all these locations, 5062 traffic Officials would be needed which constitutes a 400% increase in current staffing level (Field Survey, 2009).

Again the Traffic Division accounts for approximately 15% of total DMP manpower and comprises one of the seven divisions. Currently no career specialization policy exists within any division of the police force Staffs are thus transferred both between districts and between divisions; this prevents the development and retention of skilled and experienced staff in the Traffic Division.

Internal Co-ordination Problem: Poor coordination and lack of representation at thana level limits the development of any accurate overview of the traffic management problems in specific zones. On the other hand, the current organizational structure befits the agency little inclined to data collection and analysis. With only aggregate accident data supplied and no traffic counts, the Traffic Division has relied on judgment for signal requests and staffing assignments.

Co-ordination Problem with DCC: DCC and RHD are responsible for construction and maintenance of the road network, BRTA is responsible for the registration of vehicles and issuing permit for roadworthiness of vehicles. But violation of traffic rules and defective smoke emitting vehicles are common phenomena in Dhaka and control of this problem is become very difficult for the lack of co-ordination of DCC, BRTA with DMP.

Wrong System Policy Problem: Habitual violators of the traffic rules need to be penalized heavily to assist with compliance with traffic rules. There is provision to increase the penalty amount for the second and subsequent violations. But at present there is no effective mechanism to detect the multiple offences in DMP.

Image Problem: Image of the DMP is not quite good to the general public and they are also suffering for the lack of respect of the citizens due to the perceived involvement corrupt practices. Through sustained effort the DMP should improve their public image as they need engage themselves in more transport dealings and launch a public awareness campaign for better enforcement of traffic rules.

3.4 Others Institutional Problems

Lack of Vision: A vision sets the direction for development and guides the formulation of policy measures and strategies to attain certain objectives. Unfortunately, no such vision for transport development exists in Dhaka city as well as Bangladesh. That is why, the city is developing without long term visionary plan and most of the has occurred on ad-hoc basis as a piecemeal solution. As a result, one project is hindering another option and creating conflict with others.

Roll of Private Sector is not Defined: The role of private sector is not defined whereas attributes of the private sector enable it to respond rapidly to market changes through speedy decision making and investment. Evidence in many countries has shown that private sector ownership and operation of transport services can also deliver social benefits to the people as a whole. In order to secure competitive access to industrialised economies and global trade generally, and also to exploit the potentials of providing transport services to the sub-region, Bangladesh needs an active participation of private sector to bring in efficiencies of service operation and access to capital.

Non-involvement of Private Sector in the Road Development: Most of the land development occurred in the city through private or individual investment. The trend of development of private sector increases several times in last few years (housing project increase 50 to 115 in the period of 1990 to 2005). Mainly property development, industries, market etc. are the main component of such private development which is the main generator of population as well as trip. But, road sector develop only by the government. So, there evolved a great imbalance between the development of other sectors and transport sector.

Parallel Jurisdiction: Due to tremendous increase in area and population and consequent need to extend public services, the government simultaneously created various authorities and handed over many of these functions to them. This sometimes curtailed vital functions of the DCC or created parallel jurisdiction. At present as many as 51 institutions are involved in various capacities in service provision and development activities in Dhaka. There is no high level coordination between agencies at central government or metropolitan level, or indeed between private and public sector agencies. Major urban functions are divided between two ministries, neither possessing lead powers for urban development.

Lack of Co-ordination between Central and Other Agencies: There is no high level coordination between agencies at central government or metropolitan level, or indeed between private and public sector agencies. Major urban functions are divided between two ministries, neither possessing lead powers for urban development. Each is charged with the responsibility for important functions. While the Planning Commission is responsible for allocating government expenditure, it does not coordinate on a level below the national level. As a consequence, there is no strategic policy leadership at metropolitan level.

Organizations like WASA, DESA, Titas Gas, and T&T Board also provide significant services. However, none of these organizations has the capacity to serve either the whole DCC territory or even most of the citizens. They all suffer from serious institutional limitations. In this study, an opinion survey was conducted among the different professionals on coordination problem between the various organizations. From the survey it is found that more that 80% respondents argues on there is little or no coordination among the service providers.

Develop by Inexperienced and Fragmented Organization: LGED is solely responsible for rural roads where have huge options to minimize any wrong decision or implementation. HDR is responsible for national and regional highway particularly in rural areas. RAJUK is responsible for planning and DCC is responsible for road maintenance and management.

From the recent road infrastructure development projects particularly the elevated road projects undertaken by the government reveals that the Mohakhali Flyover is constructed under the supervision of RHD, the Khilgaon interchange being constructed by LGED, the proposed 3.8 km Hatirjheel Viaduct is given to DCC for implementation and Kuril Interchange is initiated by RAJUK. Recently ministry of environment is coming to BRT project, Bangladesh Bridge Authority (BBA) in flyover project , JICA in MRT project.

Lack of Transport Impact Assessment (TIA): As EIA and TIA are very specialized studies, these should be evaluated by a special technical team having expertise in environmental aspects as well as transpiration modeling and traffic management. Moreover, unlike EIA, TIA is not used to approve or disapprove a project rather findings of TIA study are used to suggest necessary adjustment regarding the building plan, parking requirements, accessibility requirements above all to suggest traffic management measures.

4. CONCLUSIONS AND RECOMMENDATIONS

Considering all organizations involved in the land use and transportation sector and their system characteristics this study has identified the following organizational problems with respect to transportation system of Dhaka City:

- Excessive number of agencies involved in the transport sector of the city.
- Disintegrated and uncoordinated actions of different urban service-providing providing agencies such as road cutting for utility services varying times.
- Overlapping jurisdiction of different agencies/ departments/ institutions.
- Lack of coordination among the sectoral agencies towards fulfillment of a comprehensive goal.
- Absence of a proper working atmosphere to operate with authority and autonomy as it is influenced by power structure (political and economic power of the society.
- Wrong development policy or faulty constitutional policy framework
- Little and/or no involvement of urban planners in formulating decisions.
- Lack of commitment of the people who are responsible to implement the plan.
- Ineffective and inefficient land use and development control by concerned authorities.
- Wide gap between plan maker, user/beneficiaries and implementing authority.
- Lack of qualified manpower, logistics and equipment
- Absence of public participation.
- Lack of long term vision and

- Poor quality non-transport urban service.
- The role of private sector is not clear.

To cope with the existing demand and to improve the performances and effectiveness of the existing organizations/institutions as well as overall transport system of the city, integrated and holistic measures should be taken encompassing with different sectors related to Dhaka transport system in different phases. Some of the short term, medium term and long term measures both in physical/infrastructural and policy measures to improve the existing institutional system are listed below:

4.1 Policy Measures

- **Setting Planning Goals:** The importance and need for planning comes only from the desire to plan. Thus the authorities and the people should be made well aware of the need for planning. Planning goals should be clearly set forth. It Is quite obvious that the "first task of planning is to establish social goals" (Islam, 2001b). The authorities must formulate a Development Plan for Dhaka city with well identified social goals. The objectives of planned urban development should be establishing social justice within the city. Planning must be for the people and not just for the privileged ones.
- As neither DCC nor RAJUK are interested to build roadway facilities by using own capital, DTCB should not only be given sole responsibility of transportation planning but also to pursue necessary funding for implementation of new roadway projects.
- Specific policy regarding future roadside development projects particularly those would be initiated by DCC and RAJUK with their own finance to adopt EIA and TIA studies before undertaking the project. Most importantly, in consideration of scarcity of road adjacent empty space as well as acute shortage of different transport facilities within the built-up areas, in the first place DCC and RAJUK should be discouraged to construct any road adjacent commercial project on the government land before ensuring road widening works, providing adequate on-street parking facilities, bus-lays, para-transit waiting place etc.
- An integrated and balanced transportation system should be developed taking into consideration the needs of the road system, non-motorized transport, public passenger transport and mass transit
- In order to avoid this situation continuing and to bring some order in to the development of Dhaka it is imperative to integrated transport and land use planning and the responsibility for such an integrated plan should be given to a single authority to ensure its success.
- There is, a need to introduce a new process and style of planning and development management at strategic. area and community level to guide the location of new land development and major infrastructure and services provision, to promote policies to increase efficiency and equity in the urban system and to guide and facilitate action by the private formal and informal sectors, Non Governmental Organizations (NGOs), Community Based Organizations (CBOs) and communities themselves.
- **Introduce of EIA and TIA Assessment:** As EIA and TIA are very specialized studies, these should be evaluated by a special technical team having expertise in environmental aspects as well as transpiration modeling and traffic management. Moreover, unlike EIA, TIA is not used to approve or disapprove a project rather findings of TIA study are used to suggest necessary adjustment regarding the building plan, parking requirements, accessibility requirements above all to suggest traffic management measures which will be required to accommodate anticipated increased traffic movements and thereby to minimizing the impact of the proposed development beforehand. Regarding these a special technical team may be attached to DTCB to give necessary advice on EIA and TIA. This arrangement will not only provide one stop service but also bring uniformity in project evaluation.
- **Control of Road Adjacent Empty Space:** According to mandated functions of DCC and RAJUK, they are allowed to construct market, shopping complex etc. for revenue generation and it is observed that so far these two organizations have built few markets and shopping centers at the very near to roadside and without considering the resulting impact on through traffic movements. In light of these there should be a specific suggestion regarding future roadside development projects particularly those would be initiated by DCC and RAJUK with their own finance to adopt EIA and TIA studies before undertaking the project. Most importantly, in consideration of scarcity of road adjacent empty space as well as acute shortage of different transport facilities within the built-up areas, in the first place DCC and RAJUK should be discouraged to construct any road adjacent commercial project on the government land before ensuring road widening works, providing adequate on-street parking facilities, bus-lays, para-transit waiting place etc.
- **Develop by Experienced Organization:** For better utilization of manpower large scale capital intensive road infrastructure projects should be developed by a single organization and in this regard RHD should be given the responsibility because of its vast experience in this regards.

- As neither DCC nor RAJUK are interested to build roadway facilities by using own capital, DTCB should not only be given sole responsibility of transportation planning but also to pursue necessary funding for implementation of new roadway projects.
- At present the permission is required from the concern authority before beginning of the project. But after the completion of the project no report is obligatory to provide to the authority which creates a chance not to follow the specification which are mandated by the authority at the time of permission. So, project completion report by the owner should be mandated and proper check according to the specification on which permission is provided is required after the completion of the project.
- Reformation of concern institution and decentralization of responsibilities and proper dissemination of resources to the local authorities should be ensured by appropriate policy; building capacity of all sectors (institutions, groups and individuals) to contribute fully to decision-making and urban development processes through proper training and exchange; and facilitating networking at all levels of different organizations.
- Controlling body or coordinating body like DTCB should be strengthen or city government could be established. The adoption of a good governance policy, incorporating transparency, accountability, predictability, and beneficiary participation are becoming the governance issues to bring in to strengthen the structure of local government for Dhaka.
- **Participatory Approach:** Participatory approach is essential for successful municipal service delivery and that will properly approach, will respond enthusiastically. One corrective step is transparency in formulating work plans. It was vigorously recommended that a bottom-up planning process and involvement of local people would result in better plan formulation, supervision, and execution.
- **Establishment of Training Institute:** The need for establishment of a training institute to take care of professional needs can hardly be overemphasized. A detailed study of the training needs and establishments of commensurate training facilities is strongly recommended. Obviously, such an institute could provide the requisite training for policy makers, professionals, stakeholders etc.
- **Establishment of a Separate Ministry:** In the present top-down approach, transport policy is developed by the responsible government agency with very little or no cognisance of other stakeholders views. Broad participation of different interest groups and consumers is essential for the effectiveness of such planning which is absent in the present practice. For effective coordination and development of an integrated transportation system in the country, all transport related ministries and their parastatals be brought under one broad based “Ministry of Transport”. The Cabinet Minister in charge of the Ministry could be assisted by several State Ministers, one each for Roads; Railways; Ports, Shipping and inland waterways; and Civil Aviation cum Tourism. Alternately, a separate ministry, through which all relevant organizations channel the formulation and implementation of their plans, could be set up to coordinate all such efforts. The system would be politically acceptable, because the minister in charge would always be a member of the party in power.

4.2 Special Suggestion for Particular Organizations

4.2.1 DTCB

- to establish a functional classification of road in consideration of access control and level of service (LOS).
- to devise a policy regarding road cutting-digging which is frequently required by different utility providing organizations; in order to minimize the necessity of frequent road cutting-digging:
- for new roadway project provision of dedicated underlying conduit system may be considered as an integral part of the project; cost of conduit construction could be recover from the potential users.
- for existing road particularly where BRT will be introduced, provision of under lying conduit system need be considered beforehand otherwise installation of utility services within the limited road width would be very critical and resulting impact of road cutting and digging would be significant.
- to plan primary and secondary road network along with to prepare side road entry plan to facilitate planned development of local road network and thereby to ensure accessibility of the localities particularly those situated behind the roadside frontage development
- not to allow further deterioration of the level of service of the existing roadway capacity by allowing indiscriminate densification of road adjacent landuse pattern
- to prepare roadway densification plan and accordingly restrict conflicting roadside development works
- to include, roadway functional classification, PCU values, saturation flows, capacity, unit rates of VOT, VOC etc. in the database.

4.2.2 RAJUK

- to revise present roadway width-based land development practice, which allows higher storied building with wider road
- to adopt access-control-based functional roadway classification system and TIA framework, which would be proposed by DTCB, for permitting road adjacent development projects
- to ensure development of well defined functional local roadway system
    - land division criteria should be such that each property would be accessible by emergency vehicles and at least a Fire brigade vehicle and an Ambulance can get in side by side
    - in case of R/A development preference should be given to organized land developer than the individual developer
- Requirements of supermarket development should be reviewed and following issues may need be included:
    - Adequate provision of operational dropping areas
    - Provision of public transport waiting facilities and future scope of incorporating direct entry with the elevated mass transit stations
    - Multi-entry based basement parking
    - Open-parking lot should be given preference over basement parking
    - TIA should assess effect of spilling queue of parking vehicles on the accessibility of nearby properties
- After construction of high-rise building, effective monitoring to ensure availability and use of the parking facility is more important than checking of parking provision in design stage. As traditionally property developer prepare two sets of plan, one for RAJUK's approval and another for construction work, this practice need to be stopped and for that a special monitoring cell may need be established along with all necessary rules and regulations. Otherwise there will be no use of following stringent rules and guidelines including EIA and TIA at the plan approval stage if it is not been translated in the field.
- Land acquisition would be a key issue for successful implementation of strategic plan and to make project cost minimum preservation of proposed r.o.w is very important.
- The recommended database should include future transport development plans to discourage construction of buildings along the proposed road alignment and to some extend to allow densification of landuse in and around the proposed alignment. This would be very useful for the planed development of city fringe areas particularly which are outside the RAJUK's jurisdictions.
- Strengthened RAJUK as Metropolitan Planning and Development Authority (MPDA) to provide a powerful command structure for both metropolitan planning and development functions at the apex of the metropolitan system, based on the existing RAJUK, with an inter-agency structure for investment coordination.
- The present RAJUK structure of Chairman/chief executive and full-time members would be retained. The permanent organization and staffing would be modified to provide a separate new Strategic Planning department for the structure planning and investment planning (MSIP) functions. A separate department is needed to maintain the identity and momentum of these functions in RAJUK.
- RAJUK should also give more specific institutional focus to policies and programs for the urban poor, preferable by creating a new department or interdepartmental task force for low-income land development and related activities, with recruitment of suitably qualified staff.

4.2.3 DCC

- to setup a dedicated traffic information broad casting center
- to include up to date inventory of roadway facilities, traffic flow map (AADT) of city road, demographic data of all Wards in the database.
- Decentralization of Power: It was observed that DCC was lacking in decentralization of power and responsibility as well as in transparency in its activities. The intention with which the 10 DCC zones were created and powers given to the zonal officers has not been realized.
- Meanwhile, the lack of transparency in the decision making and service delivery processes has resulted in a situation where citizens are unwilling to pay taxes and user charges. Remedial recommendations include:
    - effective delegation of power to the zones,
    - delivery of services through the zonal offices, and
    - transparency in DCC activities through involvement of citizens in both work plan formulation and execution.

- Citizen involvement could also be promoted by pushing some of the DCC activities down to the ward level. DCC has 90 directly elected ward commissioners, who as members of the council have both policy-making and input monitoring roles. However, by giving them responsibility for monitoring quality of services at the ward level and the resources to do this effectively, service delivery in the city could be significantly improved. Existing facilities for orienting and training the newly elected ward commissioners are inadequate and the ward commissioners are exposed only to very limited and fragmented pieces of knowledge. The ward commissioners also lack an established office, adequate staff, and a budget allocation. The strengthening of the ward commissioners is strongly recommended.
- The need to computerize, especially as regards MIS capability, is enormous. DCC cannot be reasonably expected to cope with the information requirements of managing a megacity in the 21st Century without greater capacity. The solution to this problem is to dramatically upgrade present capacity as soon as possible by special training and familiarizing and using latest advanced technology.
- **Strengthened Zonal offices:** For the sake of avoiding congestion and nondelivery of services, and also to keep DCC presence within easy reach of the people, it is essential that zonal offices be strengthened through delegation of well-defined administrative and financial power. Up to a certain limit the zonal officers should be able to work and operate on their own.
- **Recruitment of Assistants:** The human resource pool in each ward commissioner's office needs to be strengthened with a suitable, capable assistant who should be well educated, able to handle visitors, and where possible, deliver the goods in the absence of the ward commissioner.
- **Identification of Permanent Offices:** Ward commissioners. offices should be located at a permanent central site. Action must be taken by the DCC to ensure that suitable office space is made available for the ward commissioner and his/her staff.

4.2.4 BRTA

- Considering aging vehicular fleet and poor maintenance practice introduction of six-monthly fitness checking could be most appropriate than the current yearly fitness practice. Accordingly, fitness centers need be increased throughout the whole country. Singapore, with its highly productive and sophisticated test system, specifies inspections every six months for public service vehicles (PSVs), heavy goods vehicles (HGVs) and taxis.
- There is a need for the introduction of computer based vehicle checking system to make the service more reliable, quicker and most importantly to eliminate subjective judgment of vehicle inspectors and thereby to reduce the scope of illicit financial practice of the fitness issuing system. Moreover, contactless inspection system needs to be introduced so that vehicle owner does not get the chance of managing vehicle inspector to get preferential treatment.
- to device special policy in order to bring NMV drivers in driving license program
- to issue reminding letter for those who will fail to renew vehicle fitness within the due date and to share the information with the DMP officials
- to ensure training of driving school instructors
- to ensure special training for NMV drivers particularly those who are illiterate
- to include information regarding bus routes in the database
- in preparation of accident reporting-form input from research organizations particularly university academics would be worth exploring

4.2.5 DMP

- Responsibility should be given to ensure comprehensive training of their staffs so that they not only properly understand the traffic rules, regulations, different traffic control devices particularly road signs and markings but also they fully comprehend the philosophy behind a traffic rule and regulation, definition of unsafe maneuvers, dangerous way of freight carrying and its consequence, well acquainted with the checklist of common vehicle fitness items etc.

5. REFERENCES

DCC, (2004), "DCC Profile", A yearly brochure of Dhaka City Corporation, 2004

DITS, (1994), "Dhaka Integrated Transport System (DITS)", Dhaka Transport Coordination Board (DTCB), Dhaka-1000.

Islam, N. (2001) "Transportation in Dhaka City, Urbanization, Urban Planning and Development and Urban Governance", A reader for student, Centre of Urban Studies, April 2001, pp 86-96.

Islam, N. (2001a), "Generalized Landuse Models and Dhaka's Commercial Structure, Urbanization, Urban Planning and Development and Urban Governance", A reader for student, Centre of Urban Studies, April 2001, pp 80-85.

Islam, N. (2001b), "Urban Planning Process: The Western Concept, Urbanization, Urban Planning and Development and Urban Governance", A reader for student, Centre of Urban Studies, April 2001, pp 47-80.

Mahmud, S. M. S. (2009), "Identifying the Deficiencies of Land-use and Transport Development in Dhaka City", M.Sc. Engineering Thesis, Department of Civil Engineering, BUET, Bangladesh.

Rahman, M. (2004), "Institutional Issues, Reforming Dhaka City Management", Bangladesh Centre for Advanced Studies, Dhaka.

STP, (2004), "Strategic Transport Plan (STP) for Dhaka", Final Report, 2006, Dhaka Transport Coordination Board.

REGIONAL ROAD SAFETY PROBLEMS: SHARING OF EXPERIENCES FROM THE LOCAL LEARNING

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ABSTRACT

Road safety is not only a national problem but also a regional problem. Area specific strategic improvement plan with co-ordination of local pleple and ensuring involvement of them in implementation is significantly important to get sustainable and tengible benefit. This paper will mainly share the learning, ideas, knowledge which have achieved from regional view exchanging as well as problem identification and overcome strategy developement programs including regional workshop, seminar etc. The methods, objectives, program details of some cases will be discussed elaborately. The importance and the framework of community based road safety program are also pointed out. At the beginning of the paper, some nature and dimension of the road safety problems with particular importance of regional varience will be highlighted.

Keywords: *Regional problem, Sharing experiences, Local learning, Framework, Dimensions*

6. INTRODUCTION

It is well established that the patterns, characteristics, dimensions as well as factors of road safety problems are varied according to the local users' characteristics, socio-economic status, level of education and awareness, professional and biological multifariousnesses and above all environmental, ecological and natural diversities of a particular area. Study revealed that bicycle accident is the biggest road safety related problem in Rajshahi region accounting around 45 percent of total road accidents in Bangladesh. Motorcycle related accidents and injuries statistics show the disproportionately higher rate in the North-East and South Bengal including Sylhet, Pautakhali and Barguna in recent years. Though, pedestrian is the mostly affected road user in all over the country, they are significantly higher in city areas, particularly in Dhaka city. It is the local users including local professionals, policy makers, academicians, community leaders, researchers, journalists, law enforcement agencies, other stakeholders as well as local civil societies, who have better understanding about the dimensions and attributes of a specific problem of their localities. So, learning from their experiences and knowledge is utmost important to acquire more accurate, precise and in-depth knowledge and to develop effective strategies. In view with the above fact, from the very beginning, Accident Research Institute (ARI) has organized several regional road safety workshops, seminars, discussion meeting at different regional levels with the support of different organizations. In this paper, the authors mainly described the sharing of the learning, ideas, knowledge which have achieved from that regional view exchanging programs. The methods, objectives, program details of some cases have discussed elaborately. The importance and the framework of community based road safety program have been also pointed out. At the beginning of the paper, the regional variance in nature and dimension of the road safety problems are highlighted considering international and local perspectives.

➢ ROAD SAFETY PROBLEM: REGIONAL AND LOCAL CONTEXT

➢ Regional Variance

Road traffic injuries and disabilities is a global problem but is not homogeneously distributed all over the world among all user groups. According to the recent World Bank facts sheet on road safety, road safety is a global public health crisis, and fast becoming an obstacle to economic development for many low- and middle-income countries (LMICs). Taken globally, road injuries killed 1.3 million people in 2010 and were the eighth leading cause of death, according to the findings of the recently published 2010 Global Burden of Disease, an unprecedented epidemiological estimate of 150 major health conditions conducted by the International Health & Metrics Institute ((The World Bank, 2013)). Some other aspects which indicate the regional variance of the accident characteristics are given below:

- Not everyone is affected equally. Among males, traffic injuries are the leading cause of death for the 15-24 year old age group, and second cause of death for young adults aged 25-39. Injury rates are also highest and rising in the poorest regions of the world, in contrast with high-income regions with a long history of road safety programs, most of which have seen fatalities and serious injuries steadily decline over the last three decades.

- Beyond the enormous personal suffering they cause, road traffic injuries place a huge strain on health care systems, and challenge development objectives. Across low- and middle-income countries (LMICs), where 90% of fatal crashes occur, losses due to traffic injuries are estimated at US$100 billion per year. At national level, this aggregate translates into losses of 1-3% of GDP, a figure higher than many LMICs receive in development assistance (The World Bank, 2013).
- Low-income and middle-income countries have the highest burden and road traffic death rates: Most (91%) of the world's fatalities on the roads occur in low-income and middle income countries, which have only 48% of the world's registered vehicles where 5098 million people or 81% of the world's population live. Approximately, 62% of reported road traffic deaths occur in 10 counties (WHO, 2009).
- The WHO African Region had the highest mortality rate, with 28.3 deaths per 100 000 population. This was followed closely by the low-income and middle-income countries of the WHO Eastern Mediterranean Region, at 26.4 per 100 000 population (Table 1). Countries in the WHO Western Pacific Region and the WHO South-East Asia Region accounted for more than half of all road traffic deaths in the world (WHO, 2004).

Table 1: Road Traffic Injury Mortality Rates (per 100,000 Population) in WHO Regions, 2002

WHO Region	**Low-income and Middle Income Countries**	**High-Income Countries**
African Region	28.3	---
Region of the Americas	16.2	14.5
South-East Asia Region	18.6	---
European Region	17.4	11.0
Eastern Mediterranean Region	26.4	19.0
Western Pacific Region	18.5	12.0

Source: *World report on road traffic injury prevention, 2004*

- Even when deaths by various violent causes are considered, road traffic accident is found to be the leading cause accounting for 33 percent, 36 percent and 56 percent of the deaths in developed, developing and Middle-Eastern countries respectively (Jacobs & Palmer, 1996).
- Road traffic deaths are predicted to increase by 83% in low-income and middle-income countries (if no major action is taken), and to decrease by 27% in high-income countries. The overall global increase is predicted to be 67% by 2020 if appropriate action is not taken (WHO, 2009)
- South Asia will record the largest growth in road traffic deaths, with a dramatic increase of 144% between 2000 and 2020. If the low-income and middle-income countries follow the general trend of the high-income countries, their fatality rates will begin to decline in the future, but not before costing many lives (WHO, 2009).
- Road traffic injuries will rise to become the third leading cause of DALYs lost.
- Road traffic injuries will become the second leading cause of DALYs lost for low-income and middle-income countries (WHO, 2009).

➢ **Local Variance:**

Inside the country, with the variation of the local users' characteristics, socio-economic status, level of education and awareness, professional and biological multifariousnesses and above all environmental, ecological and natural diversities of a particular area, the patterns, characteristics, dimensions as well as factors of road safety problems are varied accordingly. In Bangladesh, according to the police reported accident statistics, occurrence of accident in the rural areas including rural sections of highways is almost double than the urban areas and in case of fatalities, this figure is three times higher. Apart from this, different areas show different spectacular characteristics of accident. Some brief descriptions of road traffic accident characteristics in Bangladesh with emphasis on local variance are given below:

- Analysis of all bicyclists involvement in Bangladesh showed that 45 percent of total bicycle accidents are taking place in the Rajshahi region which is significantly higher than any other division in Bangladesh (Dhaka 25 percent, Chittagong 8 percent, Khulna 15 percent, percent, Barisal 2 percent and Syllhet 5 percent). Studies also show that the child fatalities are significantly high in the Rajshahi Metropolitan city accounting for about 22 percent of the total fatalities in comparison to an average of 18 percent of the total fatalities for the six metropolitan cities (Figure 1).

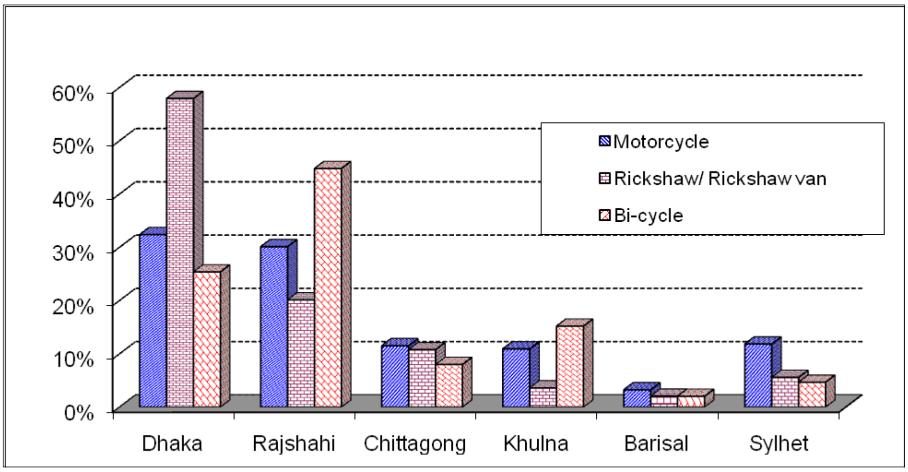

Figure 1: Involvement of Light Vehicles in Accident in Divisional Region

- Motorcycle related accidents and injuries statistics show the disproportionately higher rate in the North-East and South Bengal including Sylhet, Pautakhali, Barguna in recent years (Figure 1).
- The accident problem on the national highway network in Bangladesh is a serious and growing concern to the community. Of the total reported accidents nearly 37 percent occurred on national highways in Bangladesh. Most of the reported accidents of the highways are fatal, accounting for about 73 percent of the total highway accidents. A study on the Identification of Hazardous Road Locations on National Highways revealed that accidents and fatalities on national highways are characterized as clustering on selected sections, identified as Hazardous Road Locations (HRLs), nearly 40 percent of accidents concentrated on around 2 percent of the highway network. (Hoque et al. 2006).
- Like many other developing countries in Asia and the Aisa-Pacific region, in Bangladesh Vulnerable Road User (VRUs) viz. pedestrians, bicyclists, rickshaws, motorcyclists and other non motorised transports and non-formal paratransits are particularly at risk and constitute a large share in the total traffic fatalities and their involvement varied between 37 and 73 percent of the road accident fatalities in the metropolitan areas with an average of 65 percent. It should be noted that, the percentage of pedestrian fatalities has significantly increased from 57 percent in 1998 to 66 percent in 2006 in urban areas. Pedestrian also accounted for nearly 56 percent in non metropolitan urban areas. Further analysis revealed that most of the pedestrians' fatalities occur while crossing the road (41%) and is closely followed by walking on the road (39%) (Hoque et al, 2008b).
- Though, pedestrian is the mostly affected road user in all over the country, they are significantly higher in city areas, particularly in Dhaka city. In Dhaka Metropolitan city, almost 75 percent of road fatalities are pedestrian alone and the share of pedestrian deaths has been increasing in recent years. Current statistics revealed a deteriorating situation in metropolitan Dhaka, with pedestrians as a proportion of road crash deaths increasing from 43 percent in 1986-87 to 74 percent in 1998-2010.
- The distribution of road fatalities across road user groups for urban and rural areas represent that nearly 80 percent of road fatalities are attributed to VRUs (viz. pedestrian, bicucles, cycle rickshaws and motor cycles) in urban areas with pedestrian being by far the largest user groups in road traffic fatalities. They are also the dominant group in rural fatalities, accounting for 65 percent. Their shares varied markedly between metropolitan cities, 60 to 85 percent (Mahmud et al., 2009).
- In the period of 1998 to 2006, at least 11472 accidents occurred in urban areas accounting for 35 percent of total accidents in the country. These accidents resulted in 6519 fatalities and 8471 injuries. Around 25 percent of the total fatalities occurred in urban areas. The data presented in the table shows some possible reporting inconsistencies in the distribution of urban-rural accidents which require further investigation. Aspects of urban accidents are examined in the following sections (Hoque et al., 2008a)
- The data shows that urban accidents are concentrated in metropolitan areas. Of the total urban accidents of Bangladesh, 82 percent are metropolitan related accidents which contribute to nearly 75 percent of

urban fatalities. It may be mentioned that nearly 30 percent of total accidents and 20 percent of total fatalities are metropolitan related, with 12 percent of the total population of Bangladesh in metropolitan areas (Hoque et al., 2008a).

- A study on Metropolitan Street accident shows that nearly 22 percent of all reported accidents in Bangladesh occurred in Dhaka Metropolitan City. Nearly 52 percent of all accidents occurred at only 9 percent (18 intersections) of the total 200 intersections where at least one accident occurred during 2001-2003 (Hoque et a., 2007a).
- It is found that the age group of young people from 25 to 59 years is most vulnerable, as compared to percentage of population percentage (57% Vs 36%). Though all the road accidents cause pain and suffering to the affected family, society and loss to the nation's resources, it is particularly more severe when the victims are young people (Mahmud et al., 2009).
- Of the total child fatalities, nearly 66 percent are male and 34 percent are female. The sex distribution of adult fatalities are 86 percent are male and 14 percent are female. This shows that female children are over represented and nearly 2.5 times higher than the adult female is (Hoque et al., 2007).
- The temporal analysis of the police reported accident data reveals that 64 percent of accidents occurred during day time (6 am to 6 pm) and 36 percent at night (6 pm to 6 am) in urban areas whereas in rural areas day time share is 75 percent. It is found that accident and fatalities remained fairly evenly distributed in day times with the peak occurrence during 10 am to 12 noon. Accidents tend to occur more on Thursdays with fairly equal distribution among the week and weekend days. Accidents peaked in the months of January and March, accounting for 9.5 and 9.5 percent respectively (Mahmud et al., 2009)
- A analysis of the fatalities per million populations in different district per year show that Dhaka, the capital of Bangladesh has concentrated the highest number of fatalities per million populations (50) followed by Narayanganj (47), Munshiganj (44.7), Syleht (43), Feni (40), Rajshai (38), Majikganj (37.9) and Faridpur (37.5) (Police Reported Maap Database, 1998 to 2009).

➢ SIGNIFICANCE OF REGIONAL SAFETY PROGRAM

From the discussion of the preceding sections, it is clear that road safety problem trigger from the local level and the dimension and characteristics are varied region to region as well as community to community.

However, it is the local users including local professionals, policy makers, academicians, community leaders, researchers, journalists, law enforcement agencies, other stakeholders as well as local civil societies, who have better understanding about the dimensions and attributes of a specific problem of their localities. So, learning from their experiences and knowledge is utmost important to acquire more accurate, precise and in-depth knowledge and to develop effective strategies. In view with the above fact, from the very beginning, Accident Research Institute (ARI) has organized several regional road safety workshops, seminars, discussion meeting at different regional levels with the support of different organizations.

With other well recognized road safety initiatives, community based road safety programme has recently gained lots of interest among policy makers and professionals in preventing road traffic accidents and injuries in many countries. The concept of safe community approach, requirements, principles and indicators are discussing in the following sections.

➢ CONCEPT OF COMMUNITY BASED ROAD SAFETY PROGRAM

Community road safety programme is a challenging endeavour aiming to reduce the incidence of road traffic accidents as a major cause of death or injury, and to improve the safety of communities, through local community participation in the design and implementation of road safety programs. The main spirit of this programme is to engage the community residents themselves in defining the communities' road safety problems as well as solutions to these problems (Rahman, 2006).

➢ Safe Community Approach'- WHO Initiative To Address Injury Problem

The formal concept of a Safe Community began at the First World Conference on Injury Prevention, held in Stockholm, Sweden in September 1989. A fundamental premise of the meeting was that community level programmes for injury prevention are the key to reducing injuries. The Manifesto for Safe Communities, the resolution of the conference, called for "urgent and effective national and international action to develop and implement Safe Communities throughout the world." Since 1989, more than 70 communities have been designated Safe Communities.

For the process of Safe Community development, the WHO Safe Community model is now recognised as an effective and long term beneficial approach to the prevention of injuries at local level. Evaluating the designated Safe Communities, it has been found that about 30 to 40 percent of injuries and accidents can be reduced through this approach. But it needs to be adopted to the cultural and socio-economic conditions and existing health set ups of individual countries. there should be greater emphasis on local and national injury surveillance and community participation that would promote community ownership. Community interventions to reduce accidents and injuries occur alongside a number of other initiatives with the same goal. They are important because they add a new dimension to the fight against a growing tool of injury in both developed and developing nations. They will not replace other initiatives rather will complement them, creating a new way of tackling the ever changing pattern of accidents and injuries and dealing with problems which have proved insoluble using traditional top down approaches by using the strength of the people to bring about necessary changes in awareness, behaviour and environment(Rahman, 2006).

- **Conditions for A Successful Community Road Safety Program**

There are certain minimum requirements in terms of institutional structures and resources at the local level and at the level of National government. While local knowledge, effective working partnerships and volunteers have to come from the local level, central government has an important role in providing general guidance, expert advice and funding support. These contributions are considered separately.

Requirements at the local level

- ***A stable representative local body***. There needs to be a body at the local level to be responsible for the carriage of the strategy (or the direction of activities where there is no strategy), to coordinate stakeholders and to interact with central government.
- ***Effective personnel*** need to be available at the effective level.
- ***Effective partnerships:*** it is essential that there be good working relationships between stakeholder organisations at the local level, and that they agree about their respective roles and responsibilities in striving for shared goals.
- ***Commitment of local resources***

Requirements at the central level

- ***Management support:*** it is essential that government provide a framework for the management of community road safety.
- ***Commitment to community road safety:*** communities need to be assured that government has a long-term commitment to community road safety, and that they will not be abandoned to their own devices after having gone to the effort of establishing a strategy and a program.
- ***Expert advice:*** National road or road safety authorities generally have expert advice available, and so can advise on which problems have potentially feasible solutions, how to go about implementing these solutions, and perhaps even to provide expert services such as running seminars.
- ***Screening process for activities:*** government should provide quality control to ensure that projects which are carried out with its financial support are directed towards well planned activities that support worthwhile community road safety objectives and that have a good chance of succeeding.
- ***Evaluation:*** The government should undertake two levels of ***process*** evaluation. It is also appropriate for the central authority to attempt ***outcome***, although the difficulties associated with this should not be underestimated.

- **Values of 'Safe Communities'**

The values which are the foundation for community based injury prevention programme are as follows:

- Individual have the right to be safe, to live in the community and remain unhurt given the state of technological development.
- Individual have the right to participate in making decisions which deal with the safety of their environment.
- the community has the right to decide what are to be the priorities for actions and what resources will be allocated to interventions.
- Individuals have the right to know what is hazardous and what are risk factors for injuries.
- The community have the right to receive skills.

- **Principles of 'Safe Communities'**

The general principles which are the basis for developing a safe community programme:

- Community Organization : A community injury control programme must be based on all relevant organization in the community, closely associated with all relevant sectors of activity.
- Epidemiology and Information: Community injury prevention must be based on sufficient epidemiological and other data to document the size and nature of the accident / injury problem. The community should be aware of possibilities for injury prevention and control and the nature of problem in the local area.
- Intervention: Community will participate in interventions which are in its own interests. Possible interventions should be acceptable, beneficial to the community and should be based on intersectorial approach.
- Decision making: Priorities for action should be based on what the community feels is most important and decisions must be made from an awareness of problems and possible solutions that are inexpensives.
- Technologies and Methods: A wide range of techniques including mass media , presentation of local data, programmes in schools and personal visits to key decision makers are necessary.

- **'Safe Community' Indicators**

- Existence of a cross sectoral group responsible for injury prevention
- Involvement of the local Community Network
- A programme covering all ages, environments and situations
- The programme must show concern for high risk groups and high risk environments and aim particularly at ensuring justice for vulnerable group.
- Those responsible must be able to document the frequency and causes of injury.
- The must be a long term programme rather than a short term project.
- Utilise appropriate indicators to evaluate processes and the effect of changes.
- Analyse the community's organization and their possibility of participation in the programme.
- Involve the health care organization in both registration of injuries and the prevention programme.
- Be prepared to involve all levels of the community in solving the injury problem.
- Disseminate experiences both nationally and internationally.
- Be prepared to contribute to a strong network of safe Communities.

Falköping, Sweden was one of the first communities to approach injury prevention with community intervention. This was not accomplished by creating a new structure, it was the result of collaborative efforts of existing organizations, associations, and welfare functions. Any community that has established a context for building relationships, organizing community intervention, and achieving results has taken the valuable first steps for becoming a Safe Community (Rahman, 2006).

- **DESIGN OF REGIONAL ROAD SAFETY WORKSHOPS**

In view with the above fact, from the very beginning, Accident Research Institute (ARI) has organized several regional road safety workshops, seminars, discussion meeting at different regional levels with the support of different organizations. Step by step procedure of the program is discussed briefly in the following sections.

- **Commencement Meeting**

At the very outset, a series of commencement meeting is organized with the local NGOs and/or local influential persons to accumulate the local resources for taking assistance to prepare of organizing committee or members, to design the overall program schedule including selection of date, venue, participants list and their address. Distribution of invitation of the participants, consent of confirmation special and chief guests, and preparation of venue are also done with the collaboration of that local body.

- **Design of Resource Materials**

The resource materials consist of preparation of technical papers, presentations and questionnaire for group discussion and opinion survey. ARI has developed with the assistance of some other experts of different organizations a comprehensive resource module for the regional workshop. This module highlight the basic understanding of the magnitude and characterises of road traffic accidents, factors of such accidents, organizational and individual roles and responsibilities to tackle this problems and community based road safety program and some guidance and example of best practices of community based road safety program.

➢ Structure of the Programs

The programs are designed to let the participants understand why road traffic accidents is a problem; why it is urgent to encounter the problem; how this is issue is hampering our social well being/structure; why should we response; and what can we do to mitigate such an epidemic? The major aim of such programs is to encourage local personnel, organizations, leaders and influential community members to react in a pragmatic way to resolve such havoc through local resources, active will, and community involvement. Almost 100 participants are participated in each program from different government, non-government organizations and civil societies including top officials of local government organizations, law enforcement agencies, NGO representatives, local academicians from different educational organizations, transport owners and workers associations members, media personnel, journalists, local political leaders, community leaders and other civil society members. Figure 2 shows a glance of the participants in the workshop.

Figure 2: A glance of participants

The tentative schedule for such a program can be as per the following (Table 2):

Table 2: Typical Regional Road Safety Workshops Program in Brief

TENTATIVE TECHNICAL PROGRAM SCHEDULE		
Time	**Item**	**Title**
9:00– 9:30	Registration	
9:30 – 10:10	Inagural Ceremony	
10:10 –10:30	Refreshment	
10:30 11:20	Presentation- 1	Road Safety Problems in Bangladesh: Dimensions and Consequences
11:20 – 12:10	Presentation - 2	Factors of Road Accident and Our Responsibilities
12:10 –1:00	Presentation - 3	Community Based Road Safety Program: Engineering, Education and Enforcement
1:00 – 2:00	Lunch and Prayer	-
2:00 – 3:00	Discussion	Group Discussion and Questionnaire Fill up
3:00 – 3:45	Presentation	Group Presentation and Preparation of Recommendations
3:45 – 4:30	Endorsement of Recommendations and Certificate Awarding &Concluding Ceremony	
4:30 – 5:00	Refreshment	

The three major components of the program are inaugural ceremony, technical session, and group/open discussion. A brief description of these sessions is given in the following.

➢ Inaugural Session

In the inaugural sessions local government administrative heads, law enforcing agency personnel, political leaders and local policy makers are invited as chief and special guests to address the audience about their understanding on the road traffic accident problem, their willingness to overcome the situation, and their commitment to the society (Figure 3).

Figure 3: Inaugural session

➢ Technical Session

The technical session usually comprises three technical papers. In the first paper the road accident problem magnitude, characteristics and trends including the area specific i.e. local peculiarities are addressed. Factors associated with this problem, mitigation policies and responsibilities of different local organizations and community personnel are elaborately narrated in the second technical presentation. Finally the model of community road safety programs, example of different countries communities' best practices, and most suitable and appropriate community based initiatives could be taken for the concerned community are pointed out and method of implementation are presented to the audiences. Figure 4 shows the technical paper presentation which are being conducted by Dr. Md. Mazharul Hoque, Founding director of Accident Research Institute (ARI) (Left) and Dr. Mohammad Mahbub Hasan Taluder, Associated professor of Accident Research Institute (ARI) (right).

Figure 4: Technical paper presentation

➢ Group Discussion

In this session, all the participants are divided in several groups relating to their expertise, responsibilities, affiliation. A questionnaire form is distributed to each group. The list of group names and the sample questions of the form are given below:

Group names:

- Road and Road Infrastructural Problem

- Education and Awareness Development
- Pedestrian Safety
- Data Collection and Database development
- HRL Problem and Correction
- Emergence Response and First Aid
- Drivers’ Behavior and Problem
- Development of Enforcement
- Local Vehicle, Problem and Solution

Sample questions:

1. Please describe briefly the problems of your locality associated with the above mentioned issues in your own language.
2. Please narrate the solutions of these difficulties/problems of the mentioned issues in detail.
3. Explain the limitations of implementing the solution approaches you have mentioned earlier.
4. Please point out the necessary plans to overcome these limitations.
5. What could be the community approach to resolve such issues? How can we implement these community approaches locally?

Figure 5: Glimpses of Group discussion

This session actually is a brainstorming and feedback session in where participants make a detailed discussion elaborately between each other and finally fill the form with their local knowledge, expertise and experience. Whole the processes are guided by the organising resource persons and experts. Figure 5 represents the glimpses of group discussion.

- **Preparation of Recommendation**

An elaborative discussion takes place on various aspects of road safety problems and priorities among the participants of the workshop. The forum lively discuss the contemporary issues of road safety both from regional and local perspective as well in order to come up with pragmatic recommendations which are subsequently compiled, summarized and documented. Then the engineering perspectives, data analysis outcomes, and social aspects are accommodated with the compiled summary of workshop outcomes to finalize the recommendation for road safety improvement of that particular community and locality. The recommendations prepared in the workshops are then conveyed to the local concerned authorities to take necessary actions. Figure 6 shows the cover page of recommendation of a regional workshop.

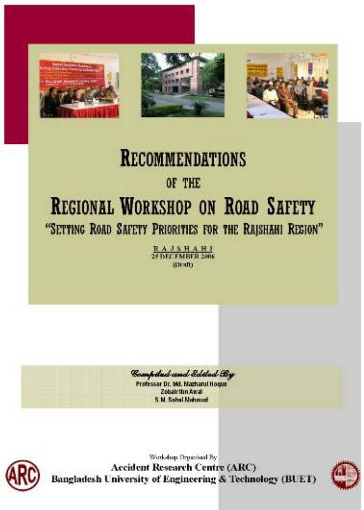

Figure 6: Cover page of recommendation of a regional workshop.

➢ CONCLUDING REMARKS

The experience of several road safety workshops have outlined the positive impact of such events in motivating local policy makers and community leaders to step forward in arresting road safety problem of their community. The workshops not only enrich their knowledge domain but also help them in taking organized efforts according to the workshop outcomes which are endorsed in a form of recommendation.

But unfortunately local bodies who appreciates such efforts primarily and provide their commitment towards the future endeavours, fails to comply with the same pace for consecutive periods. Lack of monitoring, follow-up workshops and seminar or discussion meetings and above all evaluation and appreciation of local organizers are the major drawbacks to promulgate or maintain the continuity of such efforts.

Appropriate resource allocation and budget constraints are arterial disorders to organize subsequent programs in different communities and even to follow-up the previous course of actions. Government as well as other development partners should come forward with strong wills to endorse such initiatives with proper resource and budget allotments which is urgently required to resist the recurrence losses and death tolls on roads for countries like Bangladesh.

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This paper has been prepared based on the materials and experiences of several regional road safety workshops, seminars, discussion meeting at different regional levels by Accident Research Institute (ARI) with the support of different organizations.

REFERENCE:

Jacobs, G. D. & Palmer, C. J. (1996), "Road Safety in the Emerging Nations," Intertraffic Middle East '96 Safety Symposium, Dubai, 24-25 November 1996.

Hoque, M. M., Rahman, M. F., Ashrafuzzaman, M., Sarkar, S. 2006. Observational Studies of Hazardous Road Locations on National Highways in Bangladesh", 22nd ARRB Group Conference, 29 October - 2 November 2006, Canberra, Australia.

Hoque, M. M., Mahmud, S. M. S. (2007), " Child Injuries Resulting From Road Traffic Accidents in Bangladesh", the 16th International Conference on Safe Communities, 11-13 June 2007, Tehran.

Hoque, M. M., Debnath P. (2007a), "Metropolitan Street Accidents in Bangladesh", the 16th International Conference on Safe Communities, 11-13 June 2007, Tehran.

Hoque M.M, Rabbi, S.H., Mahmud, S.M.S., Siddiqui, C.K.A., and Anowar, S.,(2007), "Road Traffic Accidents and Injuries: A Serious Safety Concern in Urban Areas of Bangladesh", Proceedings of the 6th International Symposium on New Technologies for Urban Safety of Mega Cities in Asia, 9-10 December 2007, Sheraton, Dhaka, Bangladesh

Hoque M. M., Mahmud S. M. S., Qazi. A. S. (2008a), "Dealing with Vulnerable Road User (VRU) Safety and Mobility in Urban Areas of Bangladesh: A Critical Sustainable Transport Development Challenge", Conference Proceedings, XIII CODATU, 12-14 November 2008, Ho-Chi Minh City, Vietnam.

Hoque, M. M., Anowar, S., Raihan M. A. (2008b), "Towards Sustainable Road Safety In Bangladesh", Conference Proceedings on Sustainable Transport for Developing Countries: Concerns, Issues and Options (STDC), August 2008, BUET, Dhaka.

Mahmud S. M. S., Hoque M. S., Qazi A. S. (2009), "Road Safety Problems in Bangladesh: Some Major Initiatives, Constraints and Requirements", Transport and Communications Bulletin (TCB) for Asia and the Pacific, Economic and Social Commission for Asia and the Pacific (ESCAP), No.79 on Road Safety, New York, 2009 Pp. 47-67. ISSN: 0252-4392, ST/ESCAP/SER.E/79, http://www.unescap.org/ttdw/Publications%5CTPTS_pubs%5Cbulletin79%5Cb79_fulltext.pdf

Rahman A. K. M. F. (2006), "Community Based Road Safety Programmes: Concept, Organization and Implementation", International Conference on Road Safety in Developing Countries, August 2006

The World Bank (2013), "Global Road Safety - Fact Sheet", Cathy Russell: (202) 458-8124, crussell@worldbank.org, Updated May 2013

WHO (2004), "World Report on road traffic injury prevention", ISBN 02 4 156260 9, World Health Organisation, Geneva

WHO (2008), "World health statistics 2008", Geneva, World Health Organization, 2008 (http.//www.who.int/whosis/whostat/2008/en/index.htm, accessed 30 January 2010

WHO (2009), "Global status report on road safety: time for action, 2009", Geneva, World Health Organization, 2009 (www.who.int/violence_injury_prevention/road_safety_status/2009)

Proceedings of the 2nd International Conference on Civil Engineering for Sustainable Development (ICCESD-2014), 14~16 February 2014, KUET, Khulna, Bangladesh, ISBN: 978-984-33-6373-2 (CD-ROM)

NON-MOTORIZED VEHICLE ACCIDENTS: ACCIDENT PRONE LOCATIONS AND ROAD FACTORS RESPONSIBLE FOR ACCIDENTS

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ABSTRACT

This paper represents a study of different locations where Non-Motorized Vehicle (NMV) accidents are frequent and primary road factors responsible for NMV accident with a particular context to Bangladesh. A sustainable transportation system should be like that where both the Motorized Vehicles and Non-Motorized Vehicles along with pedestrians can move safely and efficiently in the road. Accidents and road safety related issues are pervasive in Bangladesh due to its vulnerable, heterogeneous and complex transport system along with notable speed differences between motorized and non motorized vehicles. Accident records reveal that in Bangladesh during 1998-2010, among the total NMV accidents, almost 67 percent accidenst resulted in fatal . For all the modes of NMVs, the number of accidents are almost equivalent in urban and rural areas (49 percent in urban and 51 percent in rural). According to road classes, most of the NMV accidents occurred in national highways (33 percent). Among the nine national highways of Bangladesh N5 is the most accident prone with 33 percent accidents. NMV accidents are also analyzed according to some road factors like road geometry, road surface condition, road surface type, road surface quality, road features and road dividers and results are presented in this study.

Keywords: *Non-Motorized Vehicles, Accident prone road locations, Road factors, Bangladesh, MAAP5.*

1. INTRODUCTION

From the dawn of the civilization transportation system has become an integral part of human life and vehicles are the inseparable part of this system. Vehicles can be divided into two categories – Motorized Vehicles (MVs) and Non-Motorized Vehicles (NMVs). The uses of different non-motorized transportation modes were prevalent from the very beginning of the history of vehicles. Especially in the third world developing Asian and African countries, NMVs constitute the lions share of trips in the total modal share. NMV modes are becoming popular in developed countries also. Bicycles, a distinguished mode of NMV are popularly used throughout the world. Because non- motorized vehicles are fuel free, eco friendly, energy efficient, economically viable, requires significantly less road space than their motorized counterparts and provide efficient door to door service for the majority of vehicular short trips. But with this increasing number of trips there is also an augmentation in safety related problems for NMVs around the world. Approximately one out of six highway fatalities in the United States is a bicyclist or pedestrian each year (Tan, 1996). Bangladesh, being a third world developing country is flooded with different popular non-motorized modes like- Rickshaws, bicycles, push carts etc. Bangladesh is considered one of the least motorized countries in the world. According to BRTA there are about 17,51,834 motorized vehicles in Bangladesh. While the number of registered rickshaws in urban area is about 7,95,741 and in rural areas the number is estimated about 1,21,297. So it reveals that about 35% of the total vehicles are non-motorized (Ahsan et al., 2012). This huge number of NMVs plays a crucial role in the road traffic characteristics as well as in accidents. Road traffic accidents, injuries and fatalities are major concerns for Bangladesh. Bangladesh has a very high road accident fatality rate with official figures indicating more than 60 deaths per 10,000 motor vehicles. Everyday around eight persons die in road accidents. (Maniruzzaman and Mitra, 2005) But the actual rate of fatality is likely to be even higher. According to World Bank, annual fatality rate from road accidents is nearly 85.6 fatalities per 10,000 vehicles. According to a study conducted by the Accident Research Centre (ARC) of BUET, it is estimated that road accidents claim on an average 12,000 lives annually and lead to about 35,000 injuries (Rahman, 2012). These accidents and injuries claim a heavy toll of losses from NMV pullers and passengers, as they are considered the most vulnerable groups in case of road traffic accidents. From 1998-2010, about 11 percent of total accidents involved NMVs. In this study, Different road locations in Bangladesh with frequent NMV accidents have been identified and the effect of road factors on NMV accidents is investigated.

2. METHODOLOGY

The accident data was collected using the Microcomputer Accident Analysis Package Five (MAAP5) software of Accident Research Institute (ARI), BUET. Primary data of these accidents were collected by police. Accident Report Forms (ARF) are then collected by ARI from range offices and district offices as hard copies and soft copies. ARI then edits both the hard and soft copies and Keeps the MAAP5 software up to date. Data represents NMV involvement in road traffic accidents in Bangladesh during the period of 1998-2010.

3. NMV ACCIDENTS IN BANGLADESH

As the no. of NMVs are huge in Bangladesh, road traffic accidents and safety related problems associated with these are also high in number. Total 41,228 road traffic accidents were reported during 1998-2010.Among these, in a significant number of accidents, NMVs were involved. The yearly comparison between NMV accidents and other vehicle accidents can be understood from the table below.

Table 1: Year wise comparison between NMV accidents and all other accidents

Year	Total no. of Accidents	All Accidents	NMV Accidents
1998	3533	3106	427
1999	3948	3496	452
2000	3970	3503	467
2001	2925	2595	330
2002	3941	3522	419
2003	4114	3694	420
2004	3566	3235	331
2005	3322	3075	247
2006	3566	3203	363
2007	3954	3565	389
2008	3800	3423	377
2009	2815	2576	239
2010	2437	2235	202
Total	45891	41228	4663

From the above data, it has been seen that among all vehicle accidents 4,663 accidents are associated with NMVs. So 11 percent road traffic accidents were involved with NMVs.

3.1 Fatality Associated With NMV Accidents

Though the involvement of NMVs in road traffic accident may not be so eye-catching (11%), but the most horrified thing is the fatality of NMV accident which is very high compared to all other vehicle. This can be presented in the following figure.

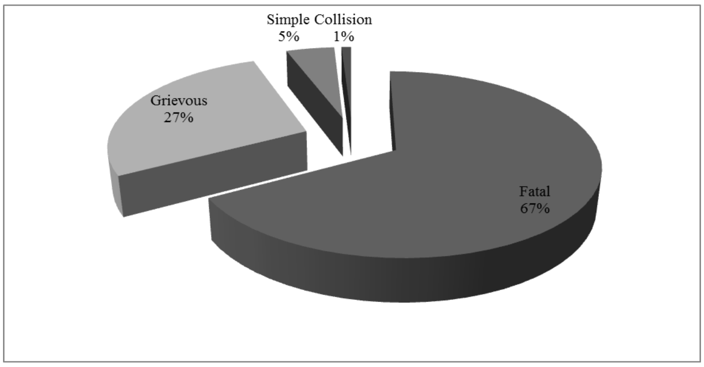

Figure 1: Fatality of NMV accidents

From the figure, it can be inferred that amongst all the NMV accidents 67 percent of the accidents are fatal. Which is more than the fatality of car accidents (30 percent) and almost equal to the fatality of heavy vehicles (approximately70 percent for both bus and truck). 27 percent NMV accidents were grievous and only 6 percent were simple and collision only accident.

3.2 Casualty Injury Related To NMV Accident

According to MAAP database during the period of 1998-2010 total 6,039 casualties occurs due to NMV accidents. Among them 3,014 people died. This means almost 50 percent people died in NMV accidents. 40 percent injuries are grievous while the rest are simple. Recording of simple injuries is diminishing day by day. In recent years the trend is seem to be decreasing.

4. NMV ACCIDENTS ACCORDING TO DIFFERENT ROAD LOCATIONS

The analysis of NMV accidents according to different locations can be divided into two parts. One is the area in which the road is situated and another is based on the class of the road.

4.1 Location Wise NMV accidents

Since non- motorized vehicles prevails throughout the country and an important para transit for the people of the Bangladesh, the accident pattern found from the data shows the accidents is not limited to a particular region but spread throughout the country. Rickshaws are the most popular form of NMV in our country. Most of the rickshaws ply in the streets of Dhaka city. If we analyze the accident data, the picture will be clearer to us.
Since most of rickshaws are in urban areas, accidents occurred are also great in number. 59 percent in urban areas compared to 41 percent in rural areas. On the other hand, Bicycle is a form of NMV that is nowadays a very popular mode of transportation in most of the countries of the world. But this vehicle is not so popular in our country, especially in city areas; this mode has not flourished at all until recently. When many city dwellers including young generations, are becoming more inclined to use bicycle for their day to day short trips. In rural areas of Bangladesh, bicycle is still a very reliable mode of transportation and is widely used. This little vehicle involved in road traffic accidents in rural areas mostly. From the accident data analyzed during 1998-2010 it has been found that out of total 1795 bicycle accidents, 1189 accidents has been occurred in rural areas while the rest are in urban areas. But when NMV accidents trend is analyzed based on whole Bangladesh, the picture found is somewhat different. Considering all the modes accident, it has been found that almost equal number of accidents occurred in both urban and rural areas.

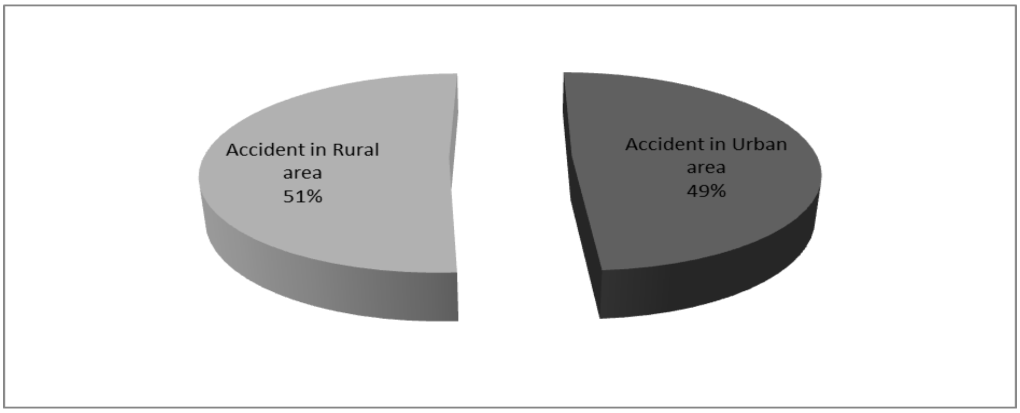

Figure 2: NMV accidents based on area

From the above figure, it can be inferred that there are almost equal number of NMV accidents occurred in both urban and rural areas during the period of 1998-2010. So the density of NMV accident is uniform and spread throughout the country.

4.2 NMV Accidents According To Road Class

According to road classes define in the ARF forms, there are five classes of roads in Bangladesh. They are national highways, regional highways, feeder roads, city roads and rural roads. NMV accidents are pervasive in all the available road classes in Bangladesh. Yearly variation of these accidents can be understood from the following figure.

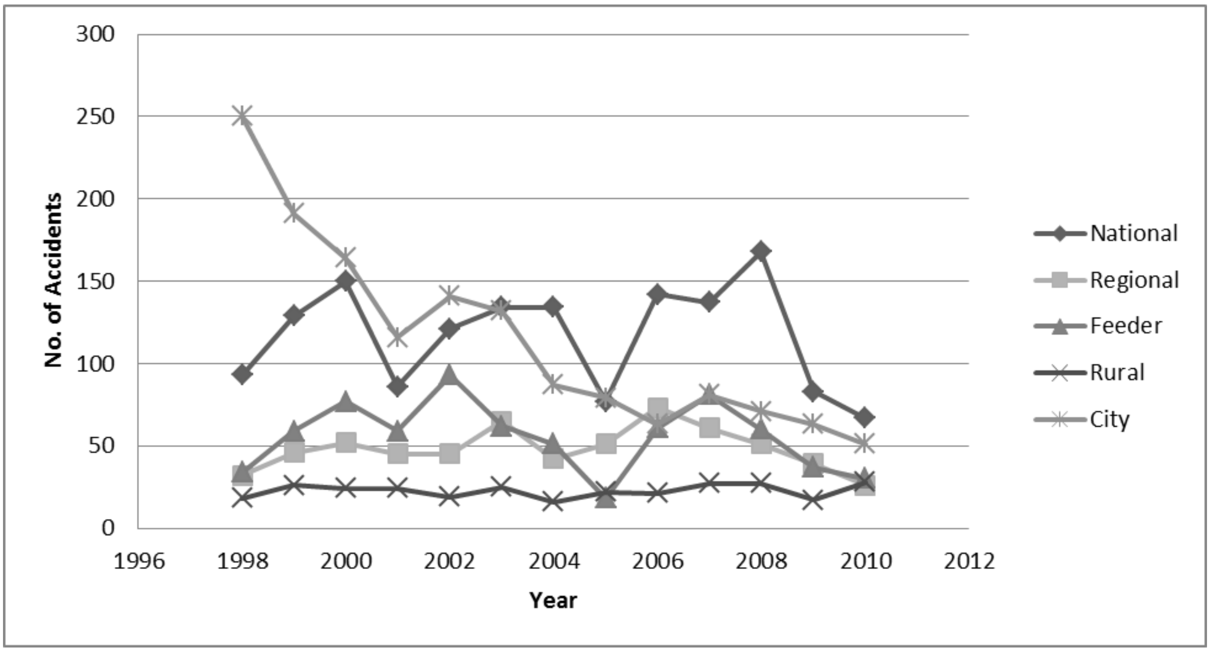

Figure 3: NMV accidents based on road class

From the figure, it can be said that up to year 2002 NMV accidents was frequent in the city roads. After that NMV accidents in city areas are decreasing and it might be due to the imposing restriction in the movement of rickshaws in most of the city areas. After 2002 national highways become the road class with highest number of NMV accidents. This is quiet unusual because very few NMVs move in the highways. It also indicates that national highways are most hazardous place for NMV movement. In rural roads least number of NMV accidents occurred. From the analysis, data indicates that in recent years NMV accidents are decreasing in all the road classes.

4.2.1 NMV accidents in national highways

From figure 3 it can be seen that the NMV accidents most predominantly occurs in the national highways of Bangladesh. At present there are nine national highways in Bangladesh. NMV accidents in the different routes of national highways are presented in figure 4.

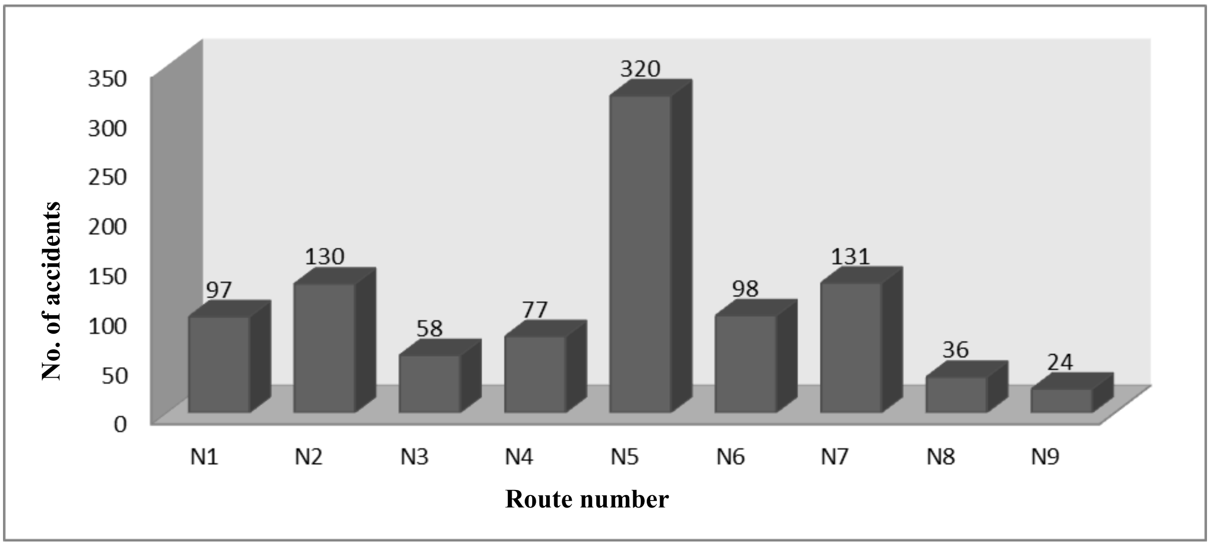

Figure 4: NMV accidents at national highways

From the above figure, It can be dictate that Route no. 5(Dhaka-Tentulia) has the highest share of NMV accidents with 33 percent of the total accidents occurred in the highways. Then comes route no. 2(Dhaka-Sylhet) and route no. 7(Dhaka-Khulna) both with a share of 14 percent NMV accidents. The main nine routes of the national highway contribute 64 percent of total national highway accidents. Rest of the branch routes contributes 36 percent.

4.2.2 NMV accidents at city roads

City streets posses the second highest no. of NMV accidents, as NMVs are mostly seen in the city roads. Out of the 1,489 NMV accidents occurred in the city areas 707 accidents are fatal, 665 accidents are grievous, 90 accidents are simple and 27 accidents are collision only accident.

4.2.3 NMV accidents at feeder roads

Feeder roads are of two types. Type A feeder roads connects thana headquarters to the arterial network and Type B feeder roads connects growth centers to RHD networks. Type A is under RHD and Type B is under LGED control. Both types of feeder roads constitute the third highest no. of NMV accidents. (Figure 3) Out of the 722 NMV accidents occurred in the feeder roads 537 accidents are fatal, 158 accidents are grievous and 27 accidents are simple.

4.2.4 NMV accidents at regional roads

Though regional highways are Third respect to the total length, it is in fourth position respect to NMV accidents by road class. . Out of the 628 NMV accidents occurred in the regional roads 463 accidents are fatal, 134 accidents are grievous, 27 accidents are simple and 4 accidents are collision only accident.

4.2.5 NMV accidents at rural roads

Rural roads are of three types named as Class 1, Class 2 and Class 3. Rural road covers 90 percent of the total road network of the country and the Local Government Engineering Department (LGED) is responsible to develop and maintain the roads. The yearly trends of NMV accidents in rural roads are presented in the figure 5.

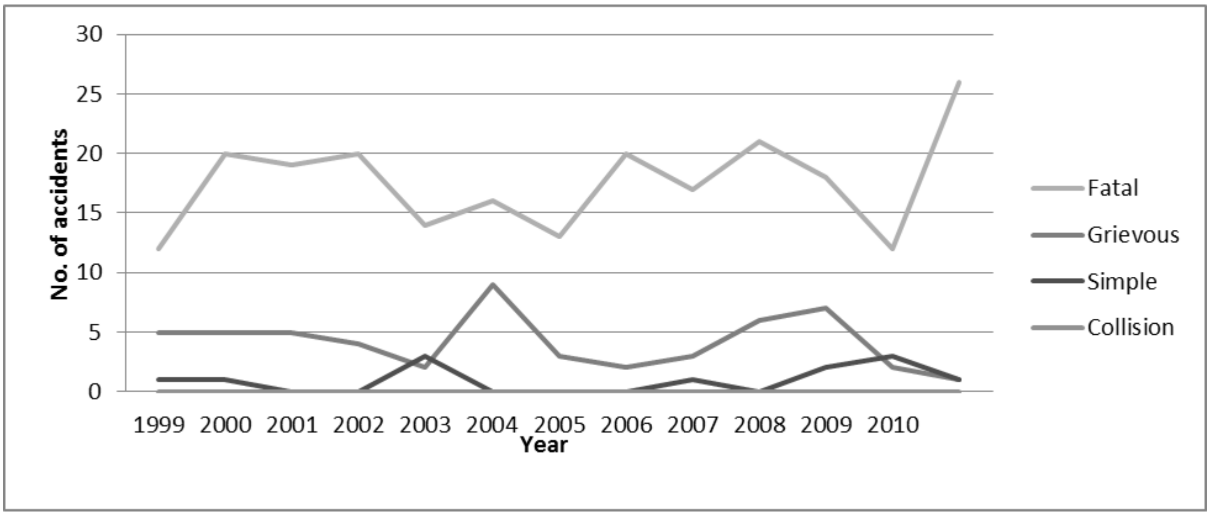

Figure 5: year wise severity of NMV accidents at rural roads

According to MAAP database, among 294 NMV accidents in the rural highways, 228 were fatal (78%). From the curve it can be seen that the fatality is in increasing trend in rural roads of Bangladesh. Highest accidents and fatality occurred in 2007-08.

5. EFFECT OF ROAD FACTORS IN NMV ACCIDENT

Road traffic may be considered as a system, in which the various components interact with each other. This system is often described as comprising three components- the human, the vehicle and the road. An accident may be considered as a 'failure' in the system. The relative contribution of human, vehicle and road factors to road accidents have been analysed in different studies. The result of one of these studies is known as the Haddon Matrix, where road factors are considered one of three responsible factors for road traffic accidents. In total, the road factors contributed to 28-34 percent of accidents (Ogden, 1996). So, road factors play a crucial role in different type of road traffic accidents. In this part of study the impact of different road characteristics on NMV accidents is described according to the data found from MAAP database.

5.1 NMV Accidents by Road Geometry

Road geometry is one of the most important factors for the analysis of accidents. NMV accidents trend by road geometry can be understood from the following pie chart.

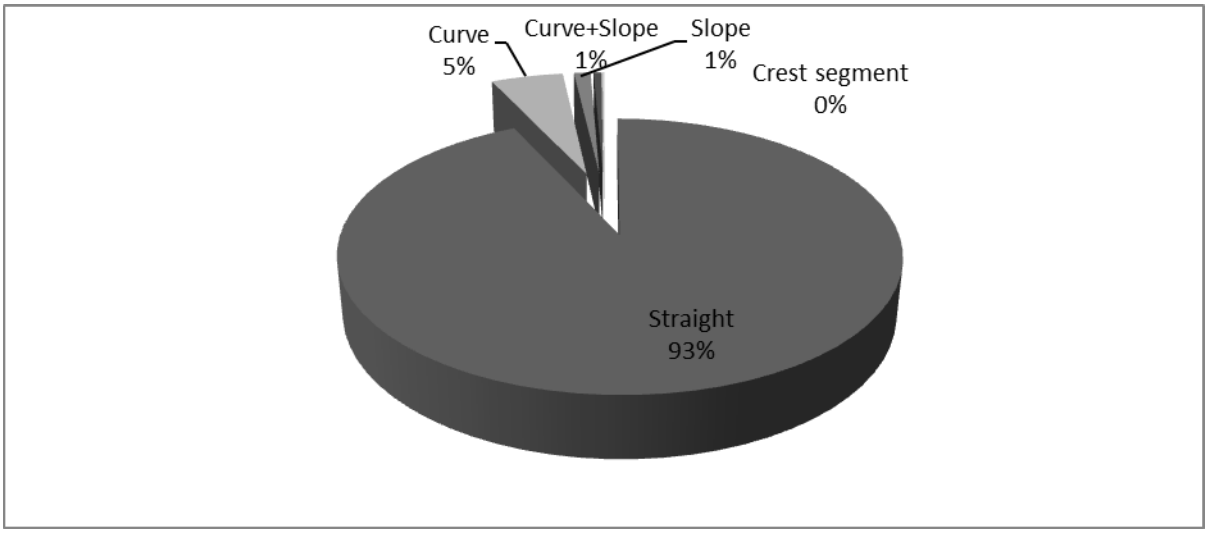

Figure 6: NMV accidents by road geometry

From the figure, it is seen that 93 percent NMV accidents occurred in straight section of road. 5 percent NMV accidents took place in curved section. Form MAAP database, in recent years (2005-2010) no. of accidents are decreasing in both straight and curved sections and increased, though very small amount in slope sections.

5.1 NMV Accidents by Road Surface Condition

Road surface condition is another important parameter of NMV accidents. NMV accidents trend by road surface condition can be understood from the following pie chart.

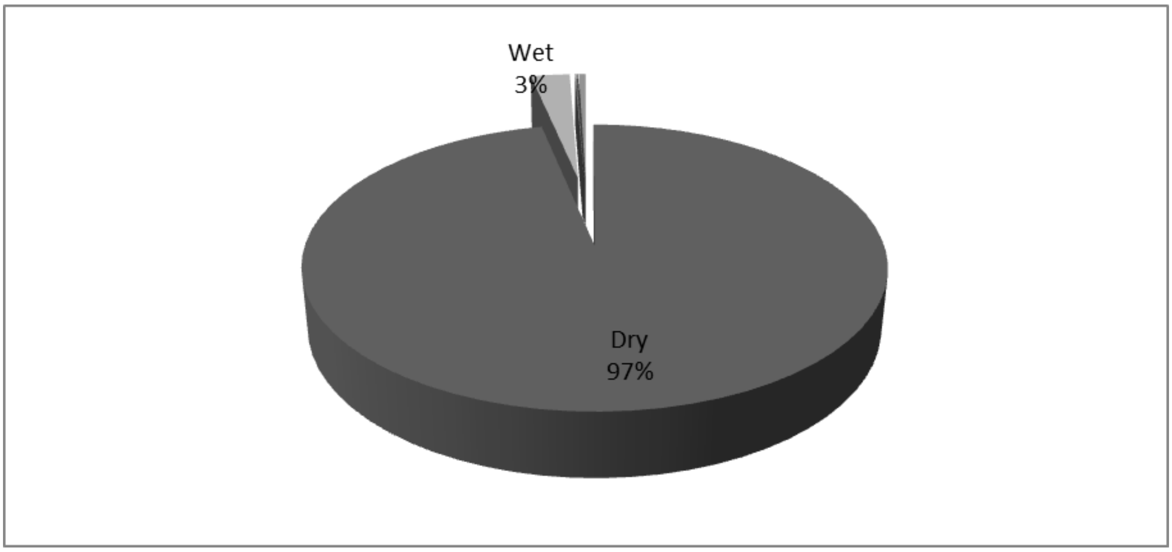

Figure 7: NMV accidents by road surface condition

From the figure, it can be said that 97 percent NMV accidents occurred when the road surface was dry. 3 percent NMV accidents took place in wet road surface. From MAAP database, in recent years (2009-2010) no. of accidents has significantly reduced in dry surface conditions but remain almost same in wet conditions.

5.2 NMV Accidents by Road Surface Type

Road surface type is another important element of NMV accidents. NMV accidents trend by road surface type can be understood from the following pie table.

Table 2: NMV accidents by road surface type (1998-2010)

Year	Surface type			
	Sealed	Brick	Earthen	Total
1998	422	2	3	427
1999	441	6	5	452
2000	453	8	5	466
2001	329	1	0	330
2002	413	5	1	419
2003	416	2	1	419
2004	325	3	2	330
2005	243	1	3	247
2006	358	3	1	362
2007	384	4	0	388
2008	368	5	3	376
2009	234	1	2	237
2010	197	2	3	202
Total	4583	43	29	4655

From the table, it can be represented that 4,583 of 4,655 (98 %) NMV accidents occurred in the sealed road surface. 2 percent of NMV accidents took place in brick road surface. In recent years (2009-2010) no. of accidents has significantly reduced in sealed and brick road surfaces but accidents have increased in earthen road surfaces, though very small in amount.

5.3 NMV Accidents by Road Surface Quality

Road surface quality is another element of NMV accidents. It is also related with the riding quality of a road surface. There are three types of road surface quality according to ARF forms. They are: good, rough and repair work going. NMV accidents trend by road surface quality can be understood from the following table.

Table 3: NMV accidents by road surface quality (1998-2010)

Year	Good	Rough	Repair work going	Total
1998	420	5	2	427
1999	432	14	5	451
2000	453	9	5	467
2001	316	9	5	330
2002	403	12	4	419
2003	404	7	7	418
2004	314	9	7	330
2005	237	3	7	247
2006	351	5	5	361
2007	371	13	4	388
2008	355	18	3	376
2009	225	10	3	238
2010	191	8	3	202
Total	4472	122	60	4654

From the table, we can see that 4,472 of 4,654 (96 %) NMV accidents occurred in the road surface of good quality. So 3 percent of NMV accidents took place in road surface of rough quality. With some fluctuations NMV accidents in good quality road surface is showing a decreasing trend in recent years (2006-2010). But the pattern of accidents in rough road surfaces remains almost same.

5.4 NMV Accidents by Road Features

Road features are another important element of NMV accidents. NMV accidents trend by road features can be understood from the following table.

Table 4: NMV accidents by road surface quality (1998-2010)

Year	None	Bridge	Culvert	Narrow	Speedbraker	Total
1998	417	7	2	1	0	427
1999	440	7	2	2	0	451
2000	450	9	0	5	1	465
2001	320	5	2	3	0	330
2002	403	7	3	4	1	418
2003	407	7	3	1	2	420
2004	311	8	3	6	1	329
2005	242	2	0	1	0	245
2006	353	7	0	1	0	361
2007	366	7	0	6	1	380
2008	357	7	2	6	1	373
2009	232	1	1	3	0	237
2010	194	6	0	1	1	202
Total	4492	80	18	40	8	4638

From the table, It cab be dictate that 4,492 of 4,638 (97 %) NMV accidents occurred in places with no special road features. 2 percent NMV accidents occurred in bridges and 1 percent in narrow road segment. In recent years NMV accidents has been increased in narrow road segments. This may due to the shifting of NMVs from main roads to the side roads in recent years.

5.5 NMV Accident Trends by Road Divider

Road divider is another important element of analyzing NMV accidents. NMV accidents trend by road dividers can be understood from the following pie chart**.**

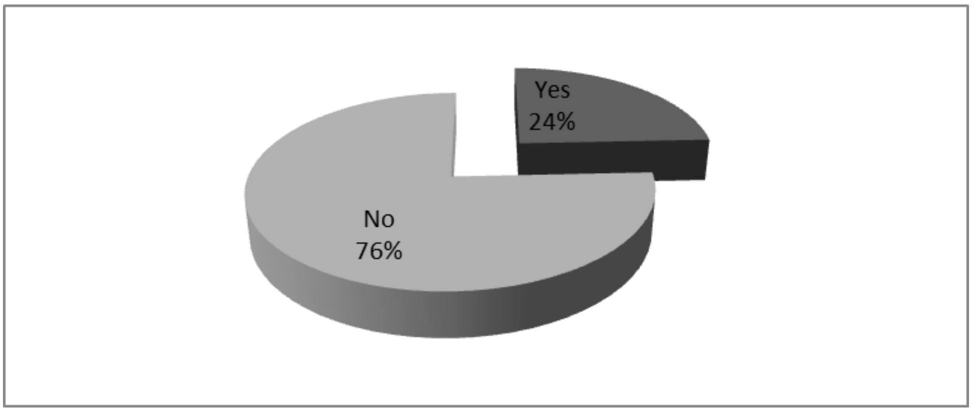

Figure 8: NMV accidents by road divider

From the figure, 76 percent NMV accidents occurred in the roadways which were not separated by any kind of road dividers. From MAAP database, Fatality of these types of accidents is 72 percent. On the other hand 24 percent NMV accident took places in divided roadways. Fatality rate is 52 percent. So it can be said that presence of a road divider in a two way road system can significantly reduced the possibilities of NMV accidents and fatalities related with these accidents.

6. RECOMMENDATIONS

Based on the research in this study some general recommendations to prevent NMV accidents are prescribed below:

- Provision of dedicated and exclusive road lanes for the NMVs especially in the highways.
- Improving the NMV design such as reducing the overall weight of NMVs, Providing gears to NMV and Adding reflectors for night time use.
- Training of the rickshaw pullers and modernizing the licensing process.
- Separate bicycle provisions especially in the busy city streets and rural highways.
- Discourage overloading of NMVs by strict regulations.
- Providing road divider in the undivided roads.
- Arrangements for proper road signs, markings and delineations to dictate bridges, narrow roads and curves ahead.

7. CONCLUSIONS

This paper mainly highlighted the accident prone road locations of Bangladesh where NMV accidents are frequent and some particular road factors which affect the characteristics of the NMV accidents. Fatalities from NMV accidents are getting more and more serious. It has found from the study that proportion of NMV accidents is almost same in rural and urban areas and most of the NMV accidents occurred in national highways of Bangladesh. Different road factors which are controlling facts in NMV accidents are also discussed in this paper. Accidents and road safety issues are now become an alarming problem of Bangladesh. With the huge numbers of NMVs plying in the streets every day, increasing accidents and safety problems make the condition even worse. The countermeasures stated in this study might be helpful in minimizing the fatality rate of NMV accidents. It is to be ensured that NMVs get their proper places to move freely and safely in the road. A reasonable balance between NMV and MV should be maintained. This can be done by modal share analysis. NMV should be given the proper level of accessibility based on its share and need for future years. Physical separation of NMV providing adequate spaces in the major arterials and busy city streets would be a very impressive solution. NMV is one of the most popular para transit in Bangladesh. Banning the NMV will not bring any solution. So concerted effort and further research is needed to make NMV a safe mode of transportation in operational, social and environmental aspect. Then Bangladesh can achieve sustainable development in transportation sector.

7. ACKNOWLEDGEMENTS

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8. REFERENCES

Ahsan, H.M., Sufian, A. and Raihan, M.A. (2012). “Road Traffic Accident Statistics and Countermeasures for NMVs In Bangladesh.” CD proceedings of 1st International Conference on Advances in Civil Engineering, CUET, Chittagong, Bangladesh.

MAAP5 Database (2010). Accident Research Institute, Bangladesh University of Engineering and Technology.

Maniruzzaman, K.M. and Mitra, R. (2005). Road Accidents in Bangladesh.

Ogden, K.W.(1996). Safer Road: A Guide to Road Safety Engineering.

Rahman,T.(2012). “Road Accidents in Bangladesh: An Alarming Issue” http://blogs.worldbank.org/endpovertyinsouthasia/node/728

Tan, C. (1996). Crash-Type Manual for Bicyclists, University of North Carolina Highway Safety Research Center.

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INVESTIGATION OF AUTOMOTIVE LIGHTING SYSTEM IN BANGLADESH

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ABSTRACT

Due to the presence of defective front head light and rear light, vehicles of Bangladesh possess open threat to road traffic safety especially at night and during rainy season. Also noticeable share of both urban and inter-district vehicles are found to ply on road without any active indicator lights. Such shocking outcomes not only signify open violation of local vehicle related laws and regulations, but also directly question the quality of vehicle maintenance practice. In this regard, one very comprehensive vehicle condition study is conducted for one year (started from December, 2011 and concluded at December, 2012) in major three bus terminals in Dhaka (Gabtoli, Sayedabad and Mohakhali bus terminals) and three truck depots (Gabtoli, Tejgaon and Kamalapur truck depot) to investigate the existing scenario of large vehicle fleet of Bangladesh. Analysis of the collected data, along with proper engineering countermeasures will be presented in the following paper.

Keywords: *Vehicle Headlight, Vehicle Indicator Lights, Maintenance of Vehicle, Vehicle Condition Survey, Bangladesh.*

1. INTRODUCTION

Transportation system is one of the most elemental components of both socio-economic and physical structure of a country. A well-planned and developed transportation system not only provides opportunities for mobility of the people but also influences the city's growth pattern and the level of economic activity (Meyer and Miller, 1984). Figure 1 shows a very simple model showing basic interactions between three essential components of a developed transportation system: persons or goods that are needed to be transported, motor vehicles, rails or water vessels that are used to move people and/or goods, and the infrastructure which includes a variety of fixed installations such as roads, streets etc. (Khisty and Lall, 2006). Thus from this relation, it is obvious that vehicles

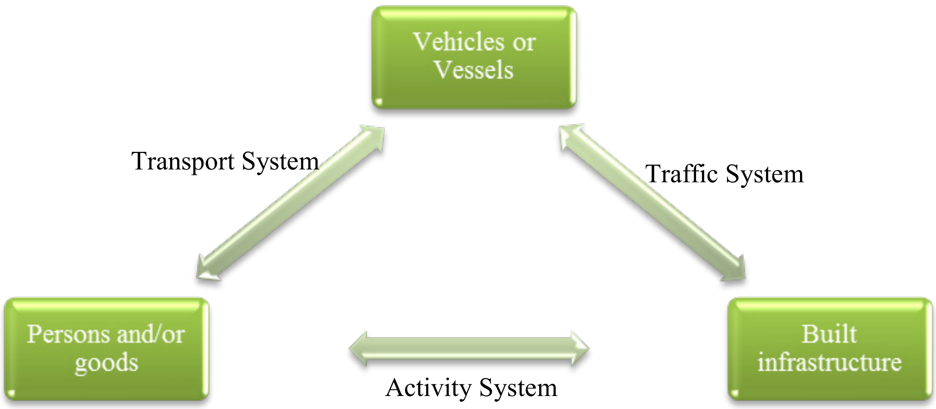

Figure 1: Basic model connecting persons, vehicles and built infrastructure

are one major important factors for an efficient transport and traffic systems. Vehicular communication system is an emerging research arena which vehicles and roadside units have to provide each other with information, such as safety warnings and traffic information. Moreover previous research on existing vehicle fleet in Bangladesh shows that, here vehicle physical conditions (including vehicle communication system) are threatened by various factors. According to drivers perception study, in Dhaka city 44% vehicle depreciation occurs due to poor traffic stop-and-go situation while for inter-city vehicles, the highest share of depreciation is due to uneven road surface (29%) (Khan and Hoque, 2012). These have direct effect on vehicle communication systems. Due to aggressive driving behaviour often the vehicle warning light and indicator lights are found faulty, even absent. Later in this paper, an attempt has made to perceive one comprehend analysis on vehicular communication system investigation.

2. METHODOLOGY

For this research, primary data collection, observations and survey are carried out during the year 2012 at different locations to obtain more representative and authentic data and results.

2.1 Vehicle Condition Survey on Communication System

A very comprehensive vehicle condition survey is conducted on the selected roads, at major bus and truck terminals in order to evaluate the roadworthiness of functional vehicle fleet by instantaneously observing vehicle communication system conditions both at stationary and running conditions and recording inventories of vehicle features after comparing with standard condition. Vehicles condition survey is conducted on 131 vehicles in total from three most important bus terminals (Gabtoli, Sayedabad and Mohakhali bus terminals) and three truck depots (Gabtoli, Tejgaon and Kamalapur truck depot) which includes both urban vehicles and inter-district vehicles. Also, night condition survey helped to further compare the data analysis with previous research result.

2.2 Photographic Survey

Captured photograph is one of the most important tools for showing the real picture of any situation. Photographs are captured from the fringe and core area of the city. Every photograph is presented with suitable caption to understand easily the relevance of captured picture.

3. ANALYSIS AND DISCUSSION

3.1 Local Vehicle Head Light Condition

Head light is the light attached to the front of a vehicle to have an artificial light the road ahead. Mainly they are of white in colour, while yellow head lights are also observed in Bangladesh. Observed are classified as follows:

- Good:
  If head lights are found in working condition providing sufficient visibility of the front road, they are tabulated as good head lights.
- Broken/Absent:
  If lights are found in non-functional working condition causing no visibility of front road or they are absent, they are termed as broken/absent head lights.
- With black strips:
  When vehicle head lights are found with black colored strip on the upper side of head lights, which minimized the glaring effect for the opposite drivers, they are termed as with black strips head lights.

Head Light Condition: Hauling-wise Analysis

After analysis of collected field data, some alarming findings are obtained regarding to head lights of vehicle. Hauling-wise survey data is shown in the following Table 1.

Table 1: Analysis of vehicle hauling-wise head light condition

Factor	Vehicle Type	Side	Good		Broken / Absent		Total		With Black Strip		Without Black Strip		Total	
			No.	%	No.	%	No.	%	No.	%	No.	%	No.	%
Head light	Short-haul vehicle	Left	80	91.95	7	8.05	87	100	32	36.78	55	63.22	87	100
		Right	82	94.25	5	5.75	87	100	39	44.83	48	55.17	87	100
	Long-haul vehicle	Left	41	93.18	3	6.82	44	100	28	63.64	16	36.36	44	100
		Right	40	90.91	4	9.09	44	100	27	61.36	17	38.64	44	100

For short haul vehicles, more vehicles are found with head lights without black strips. This provokes the situation of high beam condition on road. Even more vehicles are observed in broken or absent condition relative to the condition in long hauling vehicles.

Head Light Condition: Category-wise Analysis

Proper analysis of the head light condition observed from field cannot be done without the category-wise survey. Total condition observed of category-wise analysis is shown in the following Table 2.

Table 2: Analysis of vehicle category-wise head light condition

Factor	Vehicle Type	Side	Good		Broken / Absent		Total		With Black Strip		Without Black Strip		Total	
			No.	%	No.	%	No.	%	No.	%	No.	%	No.	%
Head light	Local Bus	Left	31	86.11	5	13.89	36	100	16	44.44	20	55.56	36	100
		Right	35	97.22	1	2.78	36	100	17	47.22	19	52.78	36	100
	Intra-city Bus	Left	28	100	0	0	28	100	7	25.00	21	75.00	28	100
		Right	27	96.43	1	3.57	28	100	12	42.86	16	57.14	28	100
	Human hauler	Left	21	91.30	2	8.70	23	100	9	39.13	14	60.87	23	100
		Right	20	86.96	3	13.04	23	100	10	43.48	13	56.52	23	100
	Inter-city Bus	Left	20	95.24	1	4.76	21	100	17	80.95	4	19.05	21	100
		Right	20	95.24	1	4.76	21	100	16	76.19	5	23.81	21	100
	Truck	Left	21	91.30	2	8.70	23	100	11	47.83	12	52.17	23	100
		Right	20	86.96	3	13.04	23	100	11	47.83	12	52.17	23	100

From the above Table 2, this is evident that except local bus, for all other vehicle types, left head light is in better condition than right one. Highest broken or absent head lights are found in trucks (Left 8.7% and right 13.04%). More than half of almost all vehicles, without inter-city buses, are found with head lights without black strips.

3.2 Local Vehicle Break Light Condition

Mainly lighting system of a motor vehicle consists of lighting and signalling devices mounted or integrated to the front, sides, rear, and in some cases the top of the motor vehicle. Red steady-burning rear brake lights, which are brighter than the rear position lamps, are activated when the driver applies the vehicle's brakes. They are required to be fitted in multiples of two, symmetrically at the left and right edges of the rear of every vehicle. The purpose of this system is to display information about driver's intentions regarding direction and speed of travel. Observed brake lights are classified as follows:

- Present (working):
  When vehicle brake lights are found fully functional while pushing the brake, they are recorded as working brake lights.
- Present (not working):
  If brake lights are found inactive or defective when pushing the brake pedals and the following vehicle drivers remain unaware about the leader’s intension of braking, they are recorded as not working brake lights.
- Absent:
  If no brake lights are found in a vehicle, they are enlisted as absent.

Break Light Condition: Hauling-wise Analysis

As vehicle condition survey included the detailed survey of the all communication factors condition detected from field, field survey data is collected and summary of hauling-wise brake light condition is shown in the following Table 3.

Table 3: Analysis of surveyed vehicle hauling-wise brake light condition

Factor	Vehicle Type	Side	Present				Absent		Total	
			Working		Not working					
			No.	%	No.	%	No.	%	No.	%
Brake light	Short-haul vehicle	Left	19	21.84	35	40.23	33	37.93	87	100
		Right	25	28.74	27	31.03	35	40.23	87	100
	Long-haul vehicle	Left	22	50.00	12	27.27	10	22.73	44	100
		Right	28	63.64	11	25.00	5	11.36	44	100

From Table 3, this has been clearly shown that more than one third of the total short haul vehicle (37.93% of left and 40.23% of right) is not equipped with active brake lightening system. In urban vehicles, which are mainly short haul vehicle, left brake light is in vulnerable condition than right. For long haul vehicles, this alarming situation goes to the extent where only half of total surveyed vehicles have functional left brake lights and almost one third vehicles have active right brake light.

Brake Light Condition: Category-wise Analysis

Detail of the brake light condition survey is collected and summary of category-wise condition survey result is shown in the following Table 4.

Table 4: Analysis of surveyed vehicle category-wise brake light condition

Factor	Vehicle Type	Side	Present				Absent		Total	
			Working		Not working					
			No.	%	No.	%	No.	%	No.	%
Brake light	Local Bus	Left	9	25.00	14	38.89	13	36.11	36	100
		Right	9	25.00	11	30.56	16	44.44	36	100
	Intra-city Bus	Left	7	25.00	9	32.14	12	42.86	28	100
		Right	10	35.71	7	25.00	11	39.29	28	100
	Human hauler	Left	3	13.04	12	52.17	8	34.78	23	100
		Right	6	26.09	9	39.13	8	34.78	23	100
	Inter-city Bus	Left	18	85.71	3	14.29	0	0	21	100
		Right	20	95.24	1	4.76	0	0	21	100
	Truck	Left	4	17.39	9	39.13	10	43.48	23	100
		Right	8	34.78	10	43.48	5	21.74	23	100

This table is nothing else but a proof of poor brake light condition in present vehicle plying on the roads in Bangladesh. Almost less than one third of all categories, except inter-city buses, are found with active brake light in both rear sides. Right brake lights are observed to be relatively more active than left one. Significant share of vehicle is detected with no brake light system. The poor brake light condition of truck with absence of left brake lights in 43.5% surveyed vehicles and right brake lights in 21.7% of total vehicles, threatens the roadway environment in national highways to a good extent.

3.3 Local Vehicle Indicator Light Condition

Indicators are blinking lamps mounted near the left and right front and rear corners or sides of a vehicle, activated by the driver on one side of the vehicle at a time to present intent to turn or change lanes toward that side of a moving vehicle. They are presently known as "Turn Light". These lights do not dazzle those who view them, and are suitably visible in conditions ranging from full darkness to full direct sunlight. They are mainly deep yellow in color. Field light condition is classified as follows:

- *Present (working):*

  If vehicle indicator lights are found active thus giving clear direction to following vehicles, they are recorded as working.

- *Present (faulty):*

  When indicator lights are found inactive or defective, thus act like a catalyst in sudden vehicular collision, they are tabulated as faulty.
- *Absent:*

  If indicator lights are absent in a vehicle, they are enlisted as absent.

Indicator Light Condition: Hauling-wise Analysis

Field survey data is collected and summary of hauling-wise indicator light condition is shown in the following Table 5.

Table 5: Analysis of surveyed vehicle hauling-wise indicator light condition

Factor	Vehicle Type	Side	Present				Absent		Total	
			Working		Faulty					
			No.	%	No.	%	No.	%	No.	%
Indicator light	Short-haul vehicle	Front Left	20	23	38	44	29	33	87	100
		Front Right	32	37	29	33	26	30	87	100
		Rare Left	31	36	32	37	24	28	87	100
		Rare Right	34	39	27	31	26	30	87	100
	Long-haul vehicle	Front Left	29	66	8	18	7	16	44	100
		Front Right	24	55	11	25	9	20	44	100
		Rare Left	27	61	6	14	11	25	44	100
		Rare Right	26	59	8	18	10	23	44	100

Relatively higher proportion of indicator light is found in working condition in long haul vehicles than short haul vehicles. Almost equal share of indicator lights for short haul vehicles is observed in active, faulty and absent category.

Indicator light Condition: Category-wise Analysis

Specific detail of the indicator light condition survey is collected. Table 6 is furnished with analyzed field data.

Table 6: Analysis of observed vehicle category-wise indicator light condition

Factor	Vehicle Type	Side	Present				Absent		Total	
			Working		Faulty					
			No.	%	No.	%	No.	%	No.	%
Indicator light	Local Bus	Front Left	11	31	17	47	8	22	36	100
		Front Right	14	39	15	42	7	19	36	63
		Rare Left	9	25	13	36	14	39	36	100
		Rare Right	7	19	15	42	14	39	36	100
	Intra-city Bus	Front Left	3	11	15	54	10	36	28	100
		Front Right	12	43	10	36	6	21	28	100
		Rare Left	10	36	10	36	8	29	28	100
		Rare Right	13	46	7	25	8	29	28	100
	Human hauler	Front Left	6	26	6	26	11	48	23	100
		Front Right	6	26	4	17	13	57	23	100
		Rare Left	12	52	9	39	2	9	23	100
		Rare Right	14	61	5	22	4	17	23	100
	Inter-city Bus	Front Left	14	67	2	10	5	24	21	100
		Front Right	13	62	2	10	6	29	21	100
		Rare Left	15	71	0	0	6	29	21	100
		Rare Right	13	62	1	5	7	33	21	100
	Truck	Front Left	15	65	6	26	2	9	23	100
		Front Right	11	48	9	39	3	13	23	100
		Rare Left	12	52	6	26	5	22	23	100
		Rare Right	13	57	7	30	3	13	23	100

From all these tabulated values, it can be easily observed that, inter district buses and trucks have relatively higher proportion of active indicator lights. Front right indicator light is relatively in better condition for local bus, human haulers and intra-district buses.

3.4 Comparison of Vehicle Night Condition Survey with Previous Research Findings

Earlier in the year 2007, vehicle night condition survey was conducted on 20 heavy trucks and 8 buses. As stated earlier, the analysis of night condition survey mainly helps to identify the vehicle communication system like head light, brake light and indicator light condition (Hasan, 2007). Following Table 7 contains the brief overview of night condition survey. Observed data shows that local buses have the maximum share of defective or absent commination system. Then inter-city bus and human haulers have a good number of defective communication systems. Among all types of communication, indicator lights are identified defective most. As overtaking of low speed vehicles is a common scenario in urban cities and national highways of Bangladesh, absence of active indicator light make the situation more critical in night time. Among other lighting system, absence of headlights in goods vehicle (trucks) turns the situation far complicated for drivers. Pedestrians are always exposed condition due to existing poor condition of vehicle lighting while crossing roads.

Table 7: Summary of vehicle night condition survey

Vehicle Factors	Perfect (%)					Absent/Defective (%)				
	Local Bus (%)	Intra-city Bus (%)	Human Hauler (%)	Inter-city Bus (%)	Truck (%)	Local Bus (%)	Intra-city Bus (%)	Human Hauler (%)	Inter-city Bus (%)	Truck (%)
Head light (Right)	77.78	85.71	91.3	95.24	82.61	22.2	14.29	8.7	4.76	13.04
Head light (Left)	86.11	89	86.96	95.24	86.96	13.9	11	13.04	4.76	17.39
Indicator Light(Front Right)	36	39	30	81	74	64	61	70	19	26
Indicator Light(Front Left)	25	32	17	76	70	75	68	83	24	30
Indicator Light(Rear Right)	19	32	57	90	61	81	68	43	10	39
Indicator Light(Rear Left)	17	29	48	86	48	83	71	52	16	52
Brake Light(Right)	19	36	57	95	61	81	64	43	5	39
Brake Light(Left)	36	18	43	95	52	64	82	57	5	48

Following Table 8 shows a comparison of observed data with previously conducted vehicle night time condition survey results.

Table 8: Summary of comparison with previous night condition research outcome

Factor	Inter-city Bus (%)				Truck (%)			
	Perfect (Recent Condition Survey)	Perfect (Previous Condition Survey)	Defective (Recent Condition Survey)	Defective (Previous Condition Survey)	Perfect (Recent Condition Survey)	Perfect (Previous Condition Survey)	Defective (Recent Condition Survey)	Defective (Previous Condition Survey)
Head light (Right)	95.24	86	4.76	14	82.61	89	13.04	11
Head light (Left)	95.24	86	4.76	14	86.96	89	17.39	11
Indicator Light(Front Right)	81	86	19	14	74	56	26	44
Indicator Light(Front Left)	76	86	24	14	70	33	30	67
Indicator Light(Rear Right)	90	86	10	14	61	33	39	67
Indicator Light(Rear Left)	86	71	16	29	48	56	52	44
Brake Light(Right)	95	86	5	14	61	86	39	14
Brake Light(Left)	95		82		52		48	

Thorough observation of this table shows that head light condition is improved for bus but for truck the condition is getting worse. With only 61% and 48% availability of rear right and rear left indicator light for trucks, vehicle manoeuvre during night is getting more accident prone. Also reduction of availability of active brake light system in trucks has become a headache for local authority.

3.5 Analysis of Photographic Survey

Following Figure 2 shows some of the striking features while conducting the research.

(a) Rear lights absent

t

(c) Faulty rear light from night condition survey

Figure 2: Vehicle Photographic Survey Findings

4. CONCLUSIONS AND RECOMMENDATIONS

From a glimpse of the complete vehicle communication system survey analysis detailed in this research, it is revealed that local vehicle suffers from significant lack of maintenance facility and they are exposed to an adverse environment. However, within the unplanned transport network in Bangladesh, vehicle performance is not up to the mark and fatal accidents are common due to defective vehicle indicator or brake lights. Thus, road unworthy, defective vehicle is one of the most important inherent limitations of the entire transport system of Bangladesh. In spite of the weakness of inadequate sample number (131 vehicles in total) the survey result signifies a representative summary of communication system condition of current vehicle fleet. Also further evaluation of diverse modes of vehicle gives a fundamental conception of relative condition of five different modes of vehicle. Besides, comparison of total survey output with previous survey result helps to identify trend of depreciation of vehicle condition. In order to minimize the existing deficiencies in vehicle communication system, some of the instant term and long term measures are listed below:

Instant Measures:

- Immediate step for supply management is necessary for road traffic system by controlling vehicular population. As with the passage of time more vehicles are found defective, instant strict regulation on number of vehicle import is required. Reduction in the number of private car and motor cycle should be balanced by increased number of public vehicles like buses.
- Random motor vehicle inspection should be introduced involving the police and vehicle inspector day and night to encourage compliance with safety standards. Only strict enforcement can make sure to keep the number of active vehicle with defective lighting system limited by fining or other degrees of punishment.
- Following steps are suggested to overcome the aggressive driving behavior of local driver:
    - Proper provision of lane based traffic movement to prevent mixing motorized and non-motorized vehicles of variable speed is needed.
    - Also time restriction of using private cars and motor cycles on road may impose a positive impression to solve present situation.
    - Side frictions like illegal parking, roadside non motor activities, non-station based boarding of vehicle of the roads contribute to the significant number of disruptions.
    - It can be said from experience that more rigorous punishments of drivers and road users will not substantially reduce such friction. Sincerity and cooperative mentality raised by repetitive social awareness campaigns will reduce the stress on road
    - Growing social awareness among local pedestrian to use footpath, foot over bridge, under pass etc. will ensure less people movement on roads. Thus sudden collision stimulated by pedestrian movement can be overcome.
- There is an urgent need to develop standard definition of observed vehicle features condition by BRTA. Absence of description of desired parameters of vehicle features at different level is seriously noticed.

Long Term Measures

- Commercial transport vehicle be standardize to fewer makes and models. Only those vehicles be allowed to import whose manufacturer can ensure regular supply in time and properly.
- The unwanted approach by local police of asking for money from vehicle drivers on active roads has created a bad influence on owners. Often it is found that owners with proper vehicle registration or fitness papers are trapped by police who demands extra cash and thus disrespect loyal vehicle owners. This provokes many owners later not to perform legal actions.
- Vehicle fitness centers need to be increased throughout the whole country. Six-monthly fitness checking could be more suitable than the current yearly fitness practice. Singapore, with its highly sophisticated test system, specifies inspections every six months for public service vehicles (PSVs), heavy goods vehicles (HGVs) and taxis. Moreover, there is need for the introduction of computer based checking system to make the service more reliable and quicker. Privatization of the fitness issuing system could be a better alternative in order to provide the service more effectively and competitively. This system has adopted in Singapore where three main contractors (all qualified to International Quality Assurance Standard ISO 9002) have been licensed. Testing of light vehicles in UK is also carried out by the private sector with appropriate control and enforcement by the government.

- To ensure smooth traffic flow, upgraded infrastructural facilities are to be provided, some of them are categorized below:
    - Dhaka is expanding outwards so new areas are to be brought under strict provision of systematic development plan. Gradually shift of capital from Dhaka may become one good consideration.
    - High occupancy vehicle like Bus rapid transit (BRT), commuter train service should be introduced to fulfill future demand.
- Availability of good quality vehicle parts is needed to be ensured. Thus control of government with skilled management and periodic monitoring will ensure easy availability of vehicle parts. Import of sub-standard products can be made easier with limited number of vehicle models. Though the perception is owners defer maintenance, but research finding shows something else. Very few owner delays to perform replacing damaged parts. If smooth market flow of required standard vehicle parts can be ensured, more longevity of vehicle features can be achieved. Also, government should give subsidy if the buses are owned by a company.

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5. REFERENCES

Hasan, M.R. (2007), “Involvement of Vehicle Factors in Road Accidents.”, M.Sc. Engg. Thesis, Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka.

Khan, S.M. and Hoque, M.S. (2012), “Vehicle Depreciation Caused by Stop-and-Go Situation in Dhaka City.” Proceedings of International Conference on Traffic and Transport Engineering, Belgrade, Serbia, 131-139.

Khisty, C.J. and Lall, B.K. (2006), “Transportation Engineering: An Introduction.” Prentice-Hall of India Private Limited, New Delhi, India, 3rd edition, pp. 22-23.

Meyer, D. and Miller, E.J. (1984), “Urban Transportation Planning: A Decision-Oriented Approach.”, McGraw Hill Series in Transportation, U.S.A., ISBN0-07-041752-0.

PEDESTRIAN FLOW CHARACTERISTICS AT WALKWAYS IN RAJSHAHI METROPOLITAN CITY OF BANGLADESH

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ABSTRACT

The walking speed of pedestrians are of prime importance in a study of function and design of pedestrian facilities. Pedestrians' speed depends on various characteristics such as age and gender of the pedestrian and width of the walkways. This paper deals with the findings of pedestrian flow characteristics on walkways in Rajshahi Metropolitan City, Bangladesh. Data were collected at three locations of walkways by using a digital camera. The data were analyzed by using the statistical software SPSSv16. The mean walking speed of the pedestrian of Rajshahi city is found 67 m/min (3.66 ft/sec.) at walkways. This speed is slower than that of Asian and American counterpart. The collected data were used to develop the speed-flow-density-space relationship of pedestrian. The maximum free-flow speed of pedestrianis is observed to be 85.26 m/min. which is higher than the Singapore, Britain and United States. Moreover, pedestrian characteristics from various cities in the world are compared. The collected data and established relationships could be used as a basis for the development of more efficient, adequate and safer facilities for the pedestrians.

Keywords: *Pedestrian, speed, flow, density, walkways*

1. INTRODUCTION

Walking is the most efficient and effective mode of transportation for short trips because every person walk sometimes in his/her every day journey. The importance of pedestrian movements is vital for economic development of city life. Rajshahi Metropolitan City (RMC) is one of the largest cities of Bangladesh. Population is increasing day by day in Rajshahi city as a result increased pedestrian's demand. The increase of population has put pressure on the pedestrian network. To identify the existing condition of the pedestrian mode, the pedestrian characteristics for various pedestrian facilities need to be investigated. Proper attention has been given to study on pedestrian behaviour and pedestrian flow characteristics for Rajshahi city. A review of different models for pedestrian facilities around the world was conducted to develop a standard model in Rajshahi metropolitan city. Various studies on walking speeds were conducted by many researchers at different areas on walkways. It was found that walking speeds of pedestrians vary over a wide range depending on the personal physical condition, gender, age and other factors. Hoel (l968) conducted a study in Pittsburgh Central Business District during peak and off -peak periods of the day and obtained a mean walking speed of 4.80 ft/sec (88 m/min). Older (1968) conducted the same study in Oxford Street, London on footways and obtained a mean walking speed of 78.6 m/min. Navin and Wheeler (1969) who investigated a study among students in University of Missouri, Columbia was found a walking speed of 79 m/min. Fruin (1971) conducted a study among commuters in United States and obtained a mean walking speed of 81 m/min. Polus *et al.* (1983) examined the movements of individual pedestrians on sidewalks in the central business district of Haifa, Israel, using a modern technique. Instead of a cine camera, a video tape recorder and a digital clock were used and obtained an average walking speed of 78.8 m/min. Tanaboriboon *et al.* (1986) conducted the first study in Southeast Asia and obtained a mean walking speed of 74 m/min. Murata (1978) presented about utilizing the traffic cell principle which introduced pedestrianisation on most of the streets in Nagano City, Japan.

In order to develop pedestrian planning standards for Rajshahi metropolitan city, it is required to conduct study on local pedestrian characteristics. The walking speed of pedestrians is the key factor for the design of pedestrian facilities. The objective of this paper is to develop the relationship between speed, flow, density and space for the pedestrians at walkways of urban areas in Rajshahi metropolitan city. This result may be useful in the planning and design of pedestrian networks in Rajshahi and can be applied to other cities in Bangladesh.

Table 1. Comparison of the Observed Walking Speeds in Different Studies

City, Country	Mean Speed (m/min)	Author(s)
(a) America and Europe		
Pittsburgh, United States	88.0	Hoel (1968)
London, England	79.0	Older (1968)
Columbia, United States	79.0	Navin and Wheeler (1969)
New York, United States	81.0	Fruin (1971)
Paris, France	87.6	Kamino (1980)
(b) Asia		
Koori-cho, Fukushima, Japan	69.6	Kamino (1980)
Osaka, Japan	90.0	Kamino (1980)
Tokyo, Japan	93.6	Kamino (1980)
Haifa, Israel	79.0	Polus et al. (1983)
Delhi, India	72.0	Gupta (1986)
Singapore	74.0	Tanaboriboon et al. (1986)
Riyadh, Saudi Arabia	65.0	Koushki (1988)
Madras, India	72.0	Victor (1989)
Bangkok, Thailand	73.0	Tanaboriboon and Guyano (1991)
Kuwait	71.0	Koushki and Ali (1993)
China	72.0	Yu (1993)
Tiruchirapalli, India	74.0	Arasan et al. (1994)
Metro Manila, Philippines	70.6	Gerilla (1995)

2. DATA COLLECTION AND METHODOLOGY

This study was conducted in the Rajshahi Metropolitan City (RMC), Bangladesh. To conduct the speed studies in the concentrated areas, three walkways were selected. Pedestrians were manually timed over a measured test length and speeds were then measured.

Table 2. Details of Study Site Locations

Walkways			
Site ID	Location of Observation Sites	Dimensions	
		Length (m)	Effective Width (m)
Ia	Rajshahi City Corporation (Walkway beside Rajshahi City Corporation)	4.00	1.10
IIa	Rajshahi University (Walkway inside Rajshahi University)	6.00	0.90
IIIa	Shaheb Bazar (Walkway at the side of Shaheb Bazar)	4.50	1.05

Thompson and Heydon (1991) introduced the concept in the United Kingdom of using slow motion video surveys to collect pedestrian data to develop speed–flow–density-space relationships. Since then, the method has been widely used, and is perceived as the preferred method for collecting pedestrian data (Zegeer *et al.* 1994). This study is based upon this methodology. The concept involves marking out an area of known dimensions (length and breadth) and recording pedestrian movements through the study area. The video tapes are then played back, and data extracted manually by an enumerator. This can take a significant amount of time, especially with large numbers of pedestrians, but the advantages are that long time periods can be analyzed; more precise measurements of data can be obtained; and there is a permanent record of events.

3. RESULTS AND DISCUSSIONS

3.1 Pedestrian Flow Characteristics in Walkways

As mentioned before, there were total three observation sites in Rajshahi metropolitan city. The observation sites are Rajshahi City Corporation, Rajshahi University, and Shaheb Bazar. The details of each location are shown in Table 2. The observed walking speed for the various types of pedestrian facilities in walkways were obtained from the video recording surveys and tabulated in Table 3.

Table 3. Pedestrians Walking Speeds on Walkways

Pedestrian Types		Characteristics				
		Sample Size	Mean Speed (m/min)	Standard Deviation (m/min)	Range	
					Low	High
Overall	Male	2048	67.24	10.44	40.35	92.74
	Female	402	65.54	10.55	40.35	91.46
	Combined	2450	66.96	10.48	40.35	92.74
Children	Male	72	62.58	9.67	42.95	86.08
	Female	27	59.20	10.82	40.87	80.65
	Combined	99	61.66	10.06	40.87	86.08
Younger	Male	1002	67.72	10.50	40.35	91.88
	Female	214	66.34	10.38	40.87	91.46
	Combined	1216	67.48	10.49	40.35	91.88
Middle Aged	Male	892	67.46	10.35	40.35	92.74
	Female	154	65.51	10.29	40.35	89.29
	Combined	1046	67.17	10.36	40.35	92.74
Older	Male	82	63.01	9.54	41.21	80.65
	Female	7	66.00	14.05	45.84	84.87
	Combined	89	63.25	9.90	41.21	84.87

It is seen in Table 3 that the mean free-flow walking speed of pedestrians was found to be 67 m/min in Rajshahi metropolitan city. The observed mean free-flow walking speed of RMC's pedestrians is comparatively slower than the Asian and American counterpart (See Table 4). It was found that the RMC's males generally walked faster than the females as their mean free-flow walking speeds are 67.24 m/min and 65.54 m/min for males and females, respectively. The mean walking speeds for children,young, middle aged and elderly pedestrians were found to be 61.66 m/min, 67.48 m/min, 67.17 m/min and 63.25 m/min respectively. Any pedestrian who appeared to be over 60 years old was termed elderly.

Table 4: Comparison of Pedestrian Walking Speeds

	ASIA			EUROPE		U.S.A.	
	Japan	Singapore	Israel	Britain	Columbia	New York	Pittsburgh
Authors	Murata (1978)	Tanaboriboon, Sim & Chin (1986)	Polus *et al.* (1983)	Older (1968)	Navin & Wheeler (1969)	Fruin (1971)	Hoel (1968)
Mean Walking Speed (m/min)	73.0	74.0	78.8	78.6	79.0	81.0	88.0

The values of pedestrian flows, pedestrian speeds, pedestrian density and pedestrian area module were computed at each study location and were analyzed by using the statistical software SPSSv16. Graphical relationship were plotted between speed and density, speed and flow, flow and density and flow and area module. These are presented in Figure 1 to Figure 3. For all locations, the analysis was done for both directions. The scattered plot of data points suggested a straight line relation between pedestrian speed and density; quadratic relationship between pedestrian flow and density, and pedestrian speed and flow and polynomial relationship between pedestrian flow and area module. The general relationships used for the analysis are developed based on single-regime approach and are described as follows:

Pedestrian speed (μ) and density (k): $\mu = a - b \times k$ (1)

Pedestrian flow (q) and density (k): $q = a \times k - b \times k^2$ (2)

Pedestrian speed (μ) and flow (q): $q = \mu(a - \mu)/b$ (3)

Pedestrian flow (q) and module (M): $q = \frac{a}{M} - \frac{b}{M^2}$ (4)

Where, speed (μ) in m/min, density (k) in ped/m^2, flow (q) in ped/m/min and Area module (M) in m^2/ped

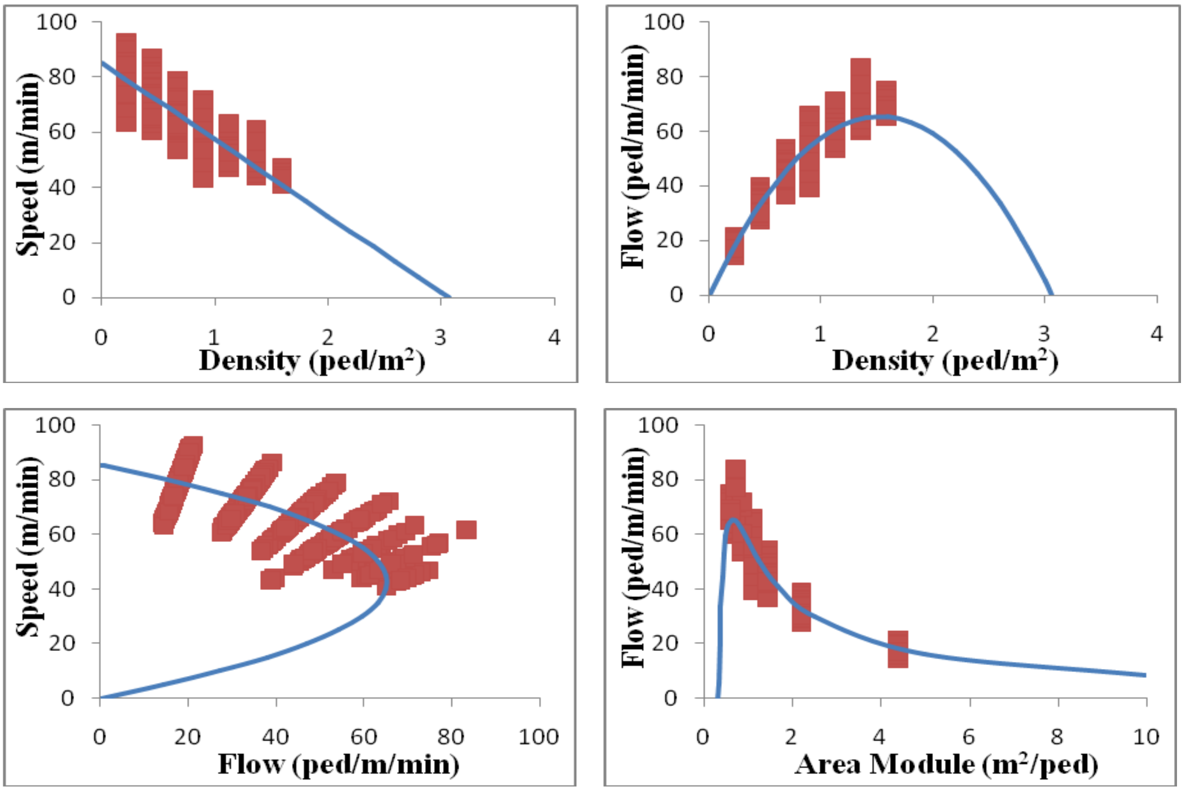

Figure 1: Pedestrian flow characteristics at Rajshahi City Corporation

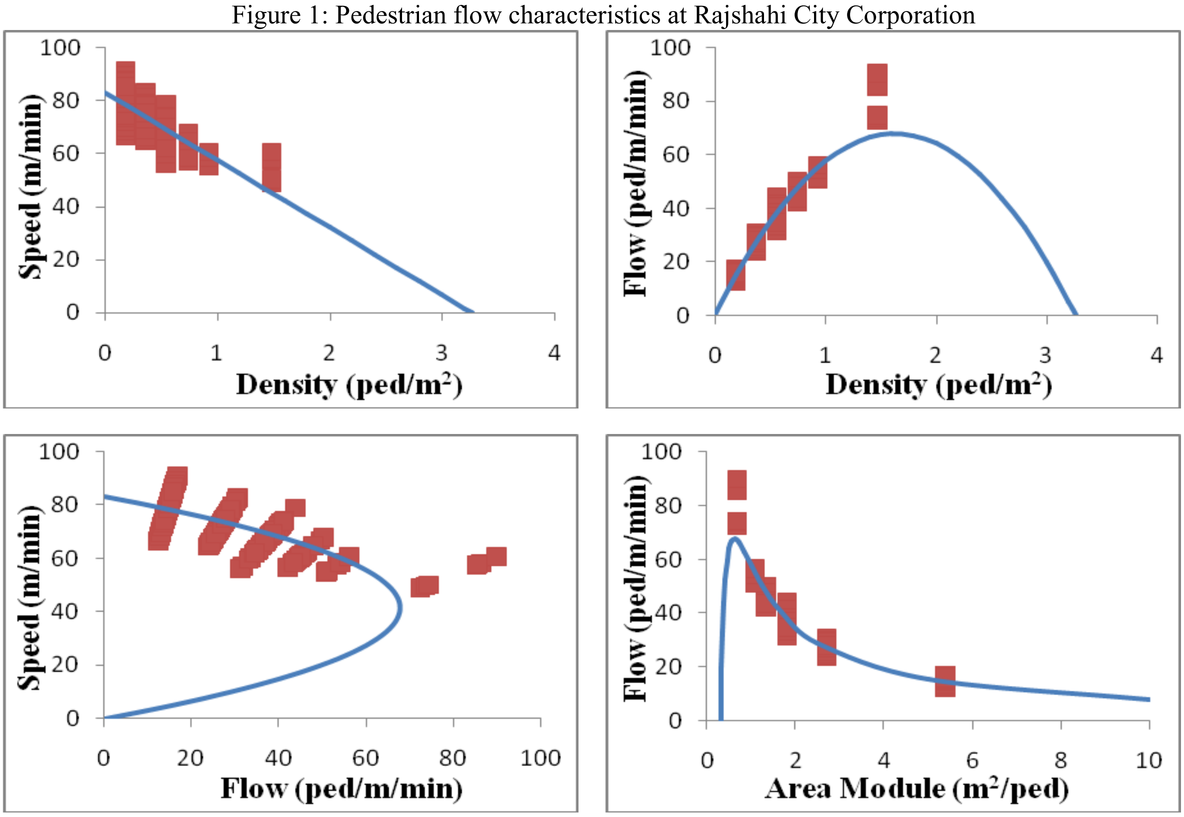

Figure 2: Pedestrian flow characteristics at Rajshahi University

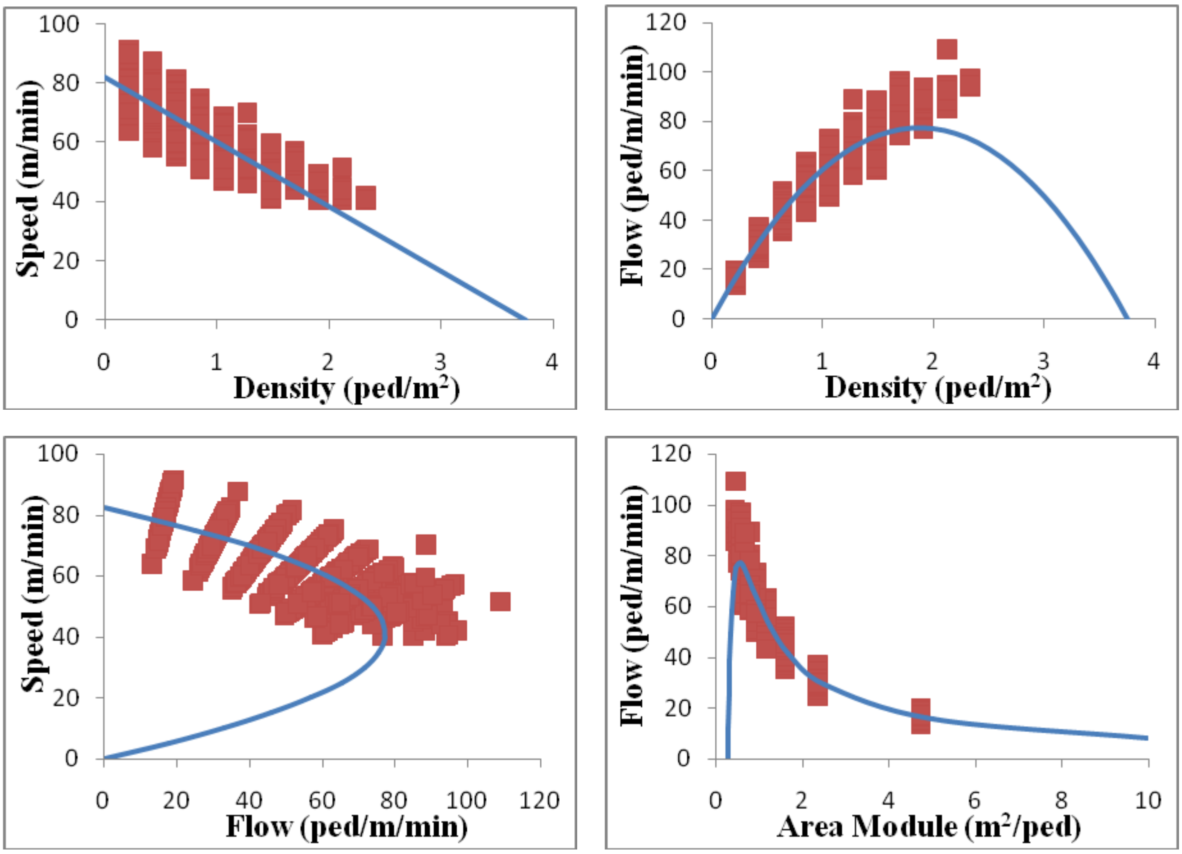

Figure 3: Pedestrian flow characteristics at Shaheb Bazar

The important flow characteristics and relationships obtained from the analysis of data are presented in Table 5. It is seen in Table 5 that the correlation coefficient R^2 varies between 0.54 and 0.97.

Table 5. Relationships Developed between Different Pedestrian Flow Characteristics for Walkways

Site ID	Location	Direction	Relationship	Model Equation	R^2 value
Ia	Rajshahi City Corporation (Walkway beside Rajshahi CityCorporation)	Mixed	Speed-density	$\mu = 85.26 - 27.81k$	0.76
			Flow-density	$q = 85.26k - 27.81k^2$	0.91
			Flow-speed	$q = \mu(85.26 - \mu)/27.81$	0.58
			Flow-space	$q = 85.26/M - 27.81/M^2$	0.85
IIa	Rajshahi University (Walkway inside Rajshahi University)	Mixed	Speed-density	$\mu = 83.17 - 25.48k$	0.63
			Flow-density	$q = 83.17k - 25.48k^2$	0.97
			Flow-speed	$q = \mu(83.17 - \mu)/25.48$	0.54
			Flow-space	$q = 83.17/M - 25.48/M^2$	0.76
IIIa	Shaheb Bazar (Walkway at the side of Shaheb Bazar)	Mixed	Speed-density	$\mu = 82.31 - 21.93k$	0.77
			Flow-density	$q = 82.31k - 21.93k^2$	0.92
			Flow-speed	$q = \mu(82.31 - \mu)/21.93$	0.62
			Flow-space	$q = 82.31/M - 21.93/M^2$	0.75

The free-flow pedestrian speeds are found to be more than 80 m/min at all of the three locations. These speeds are highest at Location Ia (85.26 m/min). This location has highest width of the carriageway (1.10 m) and the pedestrian face little frictions at this location. In case of Location IIa and IIIa, the friction due to parked vehicles is present. Due to high pedestrian flow, many pedestrians use carriageway. The friction imposed by motorized vehicles is higher at Location IIIa as compared to the Location Ia and IIa. This has resulted in a reduction in speed at the location IIa (83.17 m/min) and IIIa (82.31 m/min) compared to Location Ia.

Table 6: Pedestrian Flow Characteristics at Different Study Locations of Walkways

Site ID	Free-flow speed (μ_f), m/min	Jam density (k_j), ped/m^2	Maximum flow rate (q_{max}), ped/m/min	Area module (M) (m^2/ped.)	
				At q_{max}	Minimum
Ia	85.26	3.07	65	0.65	0.33
IIa	83.17	3.26	68	0.61	0.31
IIIa	82.31	3.75	77	0.56	0.27

It is seen in Table 6 that the maximum density was observed at Location **IIIa**, 3.75 ped/m^2 or 30 pedestrian in an area of 8 m^2. The minimum density was observed as 3.07 ped/m^2 at location **Ia** because of less frictions on roads and the high roadway width. The higher level of friction and lesser roadway width make the pedestrians to use restricted road space, thus resulting in higher density. The maximum and minimum flow rates were observed as 77 ped./m/min or 4630 ped./h and 65 ped./m/min or 3900 ped./h. It is highest at location **IIIa** and lowest at location Ia. Flow rate is lowest at location **Ia** due to open area and pedestrian freedom to use the space. The minimum area module was observed between 0.27 to 0.33 m^2/ped. and the area module at maximum flow rate is found between 0.56 and 0.65 m^2/ped. The comparison of different flow characteristics is shown in Table 7.

Table 7: Comparison of Pedestrian Flow Characteistics from Different Studies

Source	Country	Free-flow speed μ_f(m/min)	Traffic jam density k_j (ped./m^2)	Maximum flow rate (q_{max}), ped/m/min
Older (1968)	Britain	78.64	3.89	78
Fruin (1971)	United States	81.40	3.99	81
Tanaboriboon et al. (1986)	Singapore	73.90	4.83	89
Present Study	Rajshahi,Bangladesh	85.26	3.75	77

It is seen in Table 7 that the free-flow speed computed in this study are higher than those observed in Britain, United States and Singapore. The maximum density (3.75 ped./m^2) observed in this study is lower than the observed density in Britain, United States and Singapore. The maximum flow rate observed in this study (77 ped./m/min) is lower than that of Britain (78 ped./m/min), United States (81 ped./m/min) and Singapore (89 ped./m/min). Because, Bangladeshi pedestrians require less personal space than others study.

4. CONCLUSIONS

This paper aims to investigate the pedestrian flow characteristics in the walkways of Rajshahi metropolitan city. Significant variations of pedestrians mean walking speed with respect to age and gender were found. The results of the pedestrian of Rajshahi metropolitan city has a slower walking speed than the American and other Asian cities in walkways. However, the maximum flow rate obtained in this study is higher than that obtained in the Asian and Western counties. This study also shows that the characteristics of the location have effect on the pedestrian flow characteristics. The relationships developed between different flow parameters i.e. speed, flow, density and area module are observed to be satisfactory for walkways. The free-flow speeds of this study are found lower than the Asian and Western countries. The observed free-flow speed and densities are found proportional to each other. The results of this paper will constitute an important contribution to an active field of research and a resource for those developing pedestrian models in practice. It is expected that the results will be highly useful in the planning and design of pedestrian networks in Rajshahi and can be applied to other cities with similar pedestrian characteristics.

5. REFERENCES

Arasan, V. T., Rengaraju, V. R., and Rao, K. V. K. (1994). "Characteristics of trips by foot and bicycle modes in Indian city." J. Transp. Eng., 120(2), pp. 283–294.

Fruin, J. (1971). Pedestrian Planning and Design, Metropolitan Association of Urban Designer and Environmental Planners, Inc., New York

Gerilla, G. P. (1995). "Proposed level of service standards for walkways in Metro Manila." J. East. Asia Soc. Transp. Stud., 1(3), pp. 1041–1060.
Gupta, R. G. (1986). "Delhi 2010 AD: Cycle—An important mode even after the 20th century." Proc., Int. Conf. on Transportation System Studies, IIT, Delhi, India, pp. 625–632.
Hoel, L.A. (1968). "Pedestrian Travel Rates in Central Business District." Traffic Engineering, Vol. 39, No. 4, pp. 10-13.
Kamino, K. (1980). "People's movement and town design." Shôkokusha, Japan.
Koushki, P. A. (1988). "Walking characteristics in Central Riyadh, Saudi Arabia." J. Transp. Eng., 114(6), 735–744.
Koushki, P. A., and Ali, S. Y. (1993). "Pedestrian characteristics and the promotion of walking in Kuwait City Center." Transp. Res. Rec. 1396, Transportation Research Board, National Research Council, Washington D.C., USA, 30–33.
Murata, T. (1978). "Creation of Pedestrian streets in city centers." Transportation Research Record, No. 683, Transportation Research Board, National Research Council, Washington, D.C., USA, pp. 16-18.
Navin, F.P.D., and Wheeler, J. (1969). "Pedestrian flow characteristics." Traffic Engineering & Control, Vol. 39, No. 9, pp. 30-36.
Older, S.J. (1968). "Movement of pedestrians on footways in shopping streets." Traffic Engineering & Control, Vol. 10, No. 4, pp. 160-163.
Polus, A., Schofer, J.L. and Ushpiz, A. (1983). "Pedestrian Flow and Level of Service." Journal of Transportation Engineering, ASCE, Vol.109, No.1, pp.46-56.
SPSS Inc. (2012). SPSS Base 16, Application Guide, U.S.A.
Tanaboriboon, Y., and Guyano, J. A. (1991). "Analysis of pedestrian movements in Bangkok." Transp. Res. Rec. 1294, Transportation Research Board, National Research Council, Washington, D.C., USA, pp. 52–56.
Tanaboriboon, Y., Hwa, S.S. and Cho, C.H. (1986). "Pedestrian characteristics study in Singapore." Journal of Transportation Engineering, ASCE, Vol. 112, No. 3, pp. 229-235.
Thompson, S. J., and Heydon, S. (1991). "Improving pedestrian conspicuity by use of a promontory." Traffic Eng. & Control, 47(8), pp. 472–478.
Victor, D. J. (1989). "Pedestrian traffic management in Indian cities." Presented at 5th World Conf. on Transportation Research, Yokohama, Japan.
Yu, M. F. (1993). "Level of service design standards for non-motorized transport in Shanghai, China." M.Eng. thesis, Asian Institute of Technology, Bangkok, Thailand.
Zegeer, C. V., Cynecki, M., Fegan, J., Gilleran, B., Lagerway, P., Tan, H.C., and Works, R. (1994). "Summary report on FHWA study tour for pedestrian and bicyclist safety in England, Germany and the Netherlands." Rep. No. FHWA-PL-95-006, Federal Highways Administration, Washington, D.C., USA.

EFFECT OF DIFFERENT TYPES OF FILLER MATERIALS ON CHARACTERISTICS OF HOT MIX ASPHALT CONTENT

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ABSTRACT

Hot mix asphalt mix design is the process of determining appropriate proportion of the materials that would give long lasting performance paving mixture during its service life. A mix designer tries to achieve these requirements through a number of tests on the mix with varied proportions of material combinations and finalizes the best one. This often involves a balance between mutually conflicting parameters. Bitumen mix design is a delicate balancing act among the proportions of various aggregate sizes and bitumen content. For a given aggregate gradation, the optimum bitumen content is estimated by satisfying a number of mix design parameters. It is a mixture of binder, aggregate, and air in different relative proportions that determine the physical properties of the mix and, ultimately, how the mix will perform as a finished pavement. Fillers play an important role in engineering properties of bituminous paving mixes. Conventionally stone dust is used as fillers. An attempt has been made in this investigation to assess the influence of non-conventional and cheap fillers such as brick dust in bitumen paving mixes. The mineral fillers, with different percentages (5%, 7% and 8%) by total weight of the mixture, used in the study were stone dust and brick dust passing No.200 sieve. Using the different types and contents of the mineral fillers, a number of trial mixes have been prepared using the Marshal Mix design procedure to arrive at asphalt concrete mixtures that fulfill the Marshal criteria. The effects of each mineral filler type on Marshal Properties of the asphalt mixtures at their respective optimum asphalt content were evaluated and possible basis for such difference in properties was discussed. It has been observed as a result of this study that bituminous mixes with these non-conventional fillers result in satisfactory Marshall Properties. The fillers used in this investigation are likely to partly solve the solid waste disposal of the environment.

Keywords: *Title, abstract, objective, results, conclusions*

1. INTRODUCTION

Various studies have been conducted on the properties of HMA using minor changes on the ingredients of the mixture. In general the main objectives of the researches were to understand in a better way the characteristics of bituminous mixtures and evaluate the effects of constituent ingredients on the performance. Among the various studies conducted, many were concerned on investigation of effects of aggregates on the bituminous mixture performance. This is as the aggregate make up 90 to 95 percent by total weight of the mixture, they are a prime suspects influencing the performance of the mixture. The research herein with concentrates and builds on the Marshall Properties of HMA mixtures prepared using different mineral fillers by type and content. This evaluation on the subject matter was conducted by comparing the traditional crushed stone mineral filler vs. brick dust. The Marshall Mix design method consists of 6 basic steps:Aggregate selection,Asphalt binder selection,Sample preparation (including compaction),Stability determination using the HveemStabilometer,Density and voids calculations & Optimum asphalt binder content selection.

2. METHODOLOGY

This study involved investigating the Marshall properties of bituminous mixtures prepared in the laboratory using two types of mineral fillers (Passing # 200 sieves) namely stone dust and brick dust. This study involves collecting of materials for the preparation of bituminous mixtures. The materials used in the mixture includes: coarse and fine aggregates, two types of mineral fillers, and asphalt binder.

Test specimens were prepared using each type of the mineral fillers with different proportion (5%, 7% and 8%) by weight in the mix. In accordance with the Marshall Mix design procedure and criteria different mixture properties were obtained and the optimum asphalt binder content was determined.

2.1 Characteristics of Materials

The aggregates used in the research were subjected to various tests in order to assess their physical characteristics and suitability in the road construction. The aggregates were obtained from various sources available in Department of Civil Engineering in KUET.

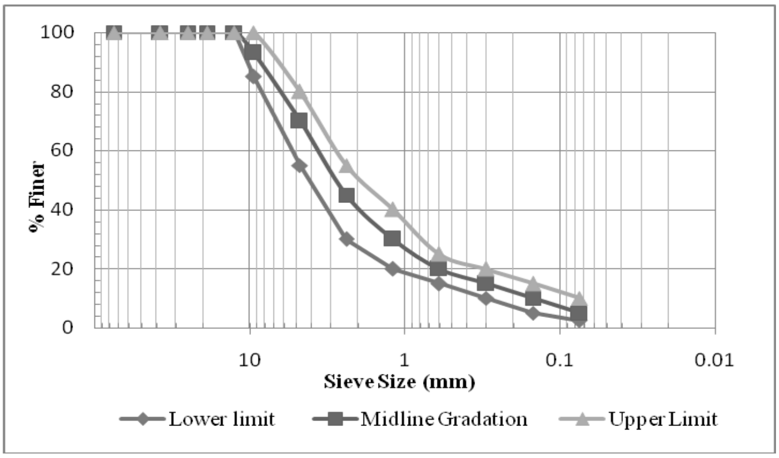

Figure 1 Aggregate gradation used in asphalt concrete Mixture

2.2 Materials test used in Paving Mixes

2.2.1 Penetration Test of Bitumen

The test was performed at 25°C. The initial reading was taken. Then the needle was released for 5 seconds and the final reading was taken the difference between the two readings gave the penetration value.

2.2.2 Softening Point of Bitumen

The test was performed by forming a sample in a brass ring, cooling it in a melting ice bath and then placing the sample within the ring in a 50°C water bath. After placing a steel ball on a sample surface, the water bath temperature was raised at the rate of 50°C per minute.

2.2.3 Marshall Test

An aggregate weighing about 1200gm and the 85/100 grade asphalt were heated to a temperature of 163°C and 130°C, respectively. Then, these ingredients were mixed at a temperature of 135°C, as previously determined. The mixture was then placed in the preheated mold and compacted using a 50 blows on either sides of the specimen. After compaction, the specimen were allowed to cool and removed from the mold by means of an extrusion jack. Using different types such as (stone dust and brick dust)and percentages (8%, 7% and 5%) of mineral fillers total 90 specimens were made. In accordance with the Marshall procedure, each compacted test specimens were subjected to determination of unit weight, void analysis, and stability and flow tests. Then, plots were made to determine values of each respective specimen prepared using different types (stone dust and brick dust) and percentages (8%, 7% and 5%) of mineral filler and optimum bitumen continents were determined.

3. ILLUSTRATIONS

3.1 Figures and Graphs

Optimum bitumen content found from the Graphs plotted from Marshall Properties of specimens made of 8% ,7% and 5% filler (stone dust) at different bitumen content (BC) are given below.

Table 1: Optimum Bitumen Content for Stone Dust

Optimum bitumen content (OBC) determination (Stone Dust)			
Filler Percentages	**8% filler**	**7% filler**	**5% filler**
Bitumen content at maximum unit weight	5.40	5.50	5.7
Bitumen content at maximum stability	5.10	5.50	5.8
Bitumen content 4 percent air voids	5.60	5.50	5.1
Optimum bitumen content	5.37	5.50	5.53

Optimum bitumen content found from the Graphs plotted from Marshall Properties of specimens made of 8% ,7% and 5% filler (Brick dust) at different bitumen content (BC) are given below.

Table 2: Optimum Bitumen Content for Brick Dust

Optimum bitumen content (OBC)determination (Brick dust)			
Filler Percentages	**8% filler**	**7% filler**	**5% filler**
Bitumen content at maximum unit weight	5.60	5.40	6.30
Bitumen content 4 percent air voids	5.50	6.00	6.40
Bitumen content at maximum stability	5.20	5.60	5.10
Optimum bitumen content	5.47	5.63	6.00

The results of Marshall Tests on bituminous mixes prepared at various filler contents by total mix of the two types of fillers. From the test results, optimum bitumen content was selected for all respective mixtures using different types and amount of mineral fillers. Table 3 indicates the properties of mixtures at their optimum bitumencontent for mixes with each filler type and content. The effect of mineral fillers on various properties of the bitumen mixtures will be discussed under subsequent sections.

Table 3. MARSHALL PROPERTIES OF BITUMINOUS MIXES AT OBC

Marshall property	Filler type	Filler content (%)		
		5	7	8
Optimum bitumen content (%)	Stone dust	5.53	5.50	5.37
	Brick dust	6.00	5.63	5.47
Air voids, (%)	Stone dust	4.00	4.00	4.00
	Brick dust	4.00	4.00	4.00
Marshall stability,(lb)	Stone dust	738.00	1695.6	1345
	Brick dust	767.84	1780.0	2050
Flow (1/100 in)	Stone dust	8.19	12.10	11.2
	Brick dust	8.89	10.98	11.8
Unit weight (pcf)	Stone Dust	136.05	136.53	137.90

	Brick dust	138.10	137.20	136.10
V.M.A (%)	Stone Dust	25.87	22.43	20.00
	Brick dust	25.97	25.25	25.15

3.2 3.2 Discussions on Test Results

3.2.1 3.2.1 Comparison of different Properties of Marshall Mixes

The following Figures represent a graphical comparison of Marshall Properties between conventional filler as stone dust and con-conventional filler as brick dust at their optimum bitumen content for mixes with each filler type and content.

Effects on optimum bitumen content

The OBC obtained using varying amount of all types of mineral fillers exhibit similar trend that is as filler content in the mixture increases, the OBC decreases. This is due to the fact that, an increased amount of filler content in the mixture fills the voids in the aggregate. This, subsequently, decreases the voids in the mineral aggregate; as a result, lower space is available for asphalt.

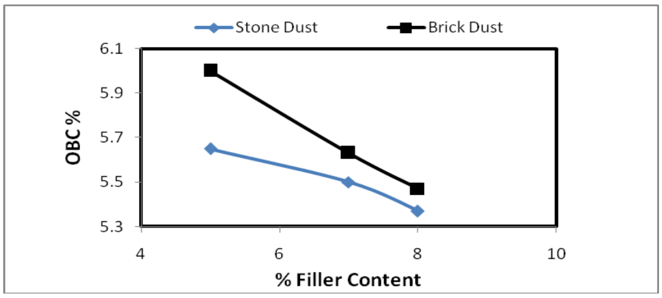

Figure 2. Effect of filler type and content on optimum bitumen content

Effects on Unit Weight

Mixes made with stone dust filler showed a trend of increase in unit weight as filler content increases and then decrease, while for mixes made with brick dust decreases as the filler content increases. The results obtained show a wide variability in unit weight for respective filler type and content, and hence it would be difficult to give an explanation on filler type effects.

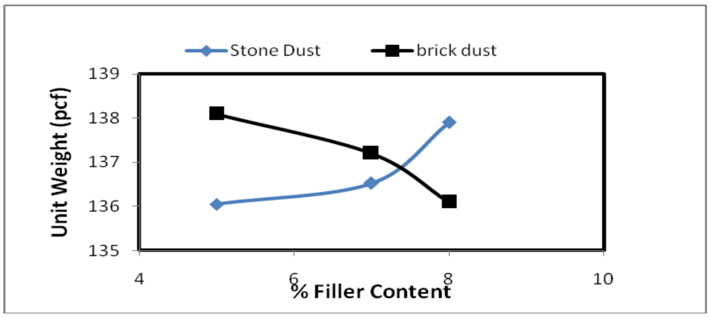

Figure2. Effect of filler type and content on unit weight at OBC

Effects on Marshall Stability

The test results obtained for individual filler type revealed that stone dust filler is finer than other types of fillers. Voids in mineral aggregate have an important role in the stiffness of mixture, that is lower values of this factor may increase the stiffness of the mixture and increases the stability.

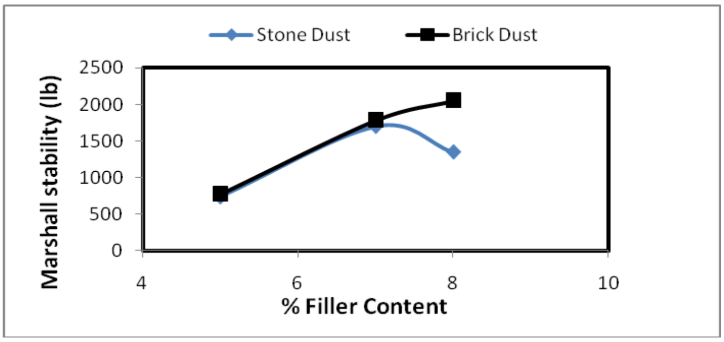

Figure 3. Effect of filler type and content on Marshall Stability at OBC

Effects on Voids in Mineral Aggregate (VMA)

As can be seen from Figure 4, mixtures using brick dust and stone dust filler types exhibit same manner, the voids in mineral aggregate keeps decreasing as the filler content in the mix increases**.**

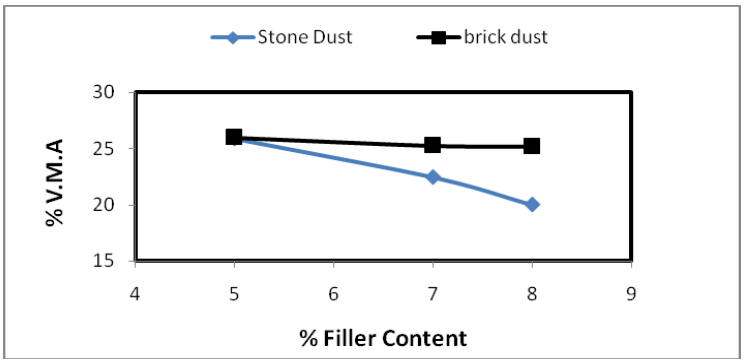

Figure 4. Effect of filler type and content on VMA at OBC

Effects on Flow

As it is clearly shown in Figure 5 below, the Flow values obtained from the laboratory prepared mixes using all filler types, meet the Marshall criteria (8 in– 14 in). At higher filler content using stone dust, lower flow values were obtained this attributes to the fact that brick dust is coarser than stone dust filler types.

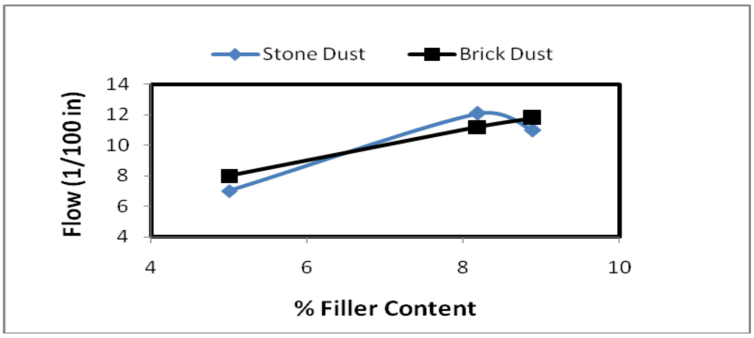

Figure 5. Effect of filler type and content on Flow Values at OBC

4. CONCLUSIONS

Mixes with stone dust and brick dust filler requires higher bitumen content that makes them to be costly from practical point of view. Whereas, mixtures prepared with stone dust and brick dust at 7% and 8% fillers, optimum bitumen content are relatively the same.

The unit weight shows a reverse relation between mixtures prepared with stone dust (5%, 7% and 8%) and brick dust (5%, 7% and 8%).

Stability values of mixes prepared with stone dust were found to be increasing up to maximum and then suddenly decrease at 8% filler content. Whereas the stability values of mixes containing brick dust keeps increasing with the filler content.

Flow values for the mixes using both brick dust and stone dust at 8% filler content are same.

5. REFERENCES

Anderson, D.A., Bahia, H.U. and Dongre, R. (1992). Rheological properties of mineral filler asphalt mastics and their relationship to pavement performance, Richard C. Meininger, (Ed).American Society for Testing Materials, Philadelphia, U.S.A., ASTM STP 1147.

Francken, L. (1998). Rilem report # 17: Bituminous Binders and Mixes, Rilem Technical Committee 152-Pbm, London.

Rahman, A. (2011). “Effect of Different Types of Fillers in Bituminous Paving Mixes”- from a thesis paper of KUET.

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MOTORCYCLE RISK ASSESSMENT IN BANGLADESH

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ABSTRACT

Motorcycle is the one of the most vulnerable motor vehicles as motorcyclists are more exposed to road traffic hazards. Evidence suggests that the risk of motorcycles is 50 times higher than cars as far as deaths and serious injuries are concerned. But motorcycles as the cheapest motorized vehicles are experiencing widespread popularity around the world. In India, motorized two wheelers comprised over 71% of registered motorized vehicles as of 2004 and continued to represent over 70% of annual vehicle sales through 2007. In Bangladesh, motorcycles occupy about 55% of all registered vehicles and registered ownership of motorcycles is increasing at an astonishing rate - more than 590% in the period between 1998 and 2012. In 2010, 23% of global road deaths were among motorcyclists costing enormously to the society. For example, fatal and serious crashes involving motorcyclists cost £1.8 billion annually in the UK alone. Motorcycle risk associated with road and roadway environment is quite significant as there has been a lack of attention to safe road infrastructure for motorcyclists. Recent iRAP (International Road Assessment Programme) assessment of 1400 km of highways in Bangladesh indicated the severity of road safety hazards for motorcyclists as the assessment revealed that 71% of assessed highways are 2-star or less (out of possible 5-star) indicating a relatively high level of risk of deaths and injuries. A critical factor in risk assessment of motorcyclists is the speed at which the motorcycle is travelling. The safe system concept emphasizes road and roadside design to reduce the risk of speed related crashes. This paper introduces the severity of motorcycle safety around the world and describes the role of road infrastructure and speed factors in motorcycle accidents. It also summarizes different motorcycle safety improvement options in terms of road infrastructure and behavioural approaches.

Keywords: *Motorcycle Safety, Risk Assessment, Road Infrastructure, Speed, Highways*

1. INTRODUCTION

The staggering increase in number of motorcycles especially in Asia is leading towards significant increase in number of accidents and therefore costing enormously in terms of losses occurred by consequent deaths and injuries. The risk becomes apparent by the fact that motorcyclists (and generally 2 wheeled motorised vehicles) together with bicycles and pedestrians have been grouped to be labelled under the term- Vulnerable Road Users (VRUs). Moreover, the safety of motorcycles and motorcyclists are not considered in developing countries like Bangladesh when current road design approach does not include any provision for this increasing number of motorcycles. The threat has been reflected in the recent iRAP (International Road Assessment Programme) road infrastructure assessment indicated that majority of the assessed length of more than one third of national highways have been estimated as very hazardous for the motorcyclists. The correlation of unprecedented increase of motorcycles and increase of accidents needs further concentration and should be considered in the future road safety action plan and management. The present paper attempts to illustrate the scenario of increasing usage of motorcycles and the severity of motorcycle safety around the world and describes the role of road infrastructure and speed factors in motorcycle accidents. It also summarizes key motorcycle safety improvement options in terms of road infrastructure and behavioural approaches.

2. THE RISK OF MOTORCYCLE RIDING

Motorcycles have noticable charactristics as they are the motor vehicles most exposed to other vehicles and road hazards and therefore the only motorized vulnerable road user group. Motorcycles are much smaller and lighter than cars, have only two wheels, and do not enclose the rider in a box of metal. These characteristics, along with others, make motorcycle riding riskier than riding in a car. As compared to car accidents, motorcycle accidents are more likely to result in death or serious injury. According to a research in UK, Motorcyclists are more than

50 times more likely to be killed than car drivers, this figure is considered per mile travelled, they are also twice as likely as pedal cyclists, the next most vulnerable vehicle group (streetdirectory.com).

Some of the risks unique to motorcycle riding include (see details at nolo.com):

- **Less visibility to cars.** Because motorcycles are smaller and more easily hidden by objects on or off the road, cars are less likely to see them, especially at intersections.
- **Road infrastructure hazards.** Things that have little effect on a car, like debris, uneven road surfaces, small objects, or wet pavement, can cause a motorcycle to crash.
- **No barrier between rider and road.** Unlike passengers in a car, bikers are not protected by a container of metal. Motorcycles also don't have seatbelts, and most don't have airbags (although manufacturers have recently introduced airbags into some models). Wearing a motorcycle helmet can offer some protection to bikers, and motorcyclists who don't wear helmets are more likely to die in an accident than those that do.
- **Less stability.** Vehicles with two wheels are less stable than those with four, especially during emergency braking and swerving.
- **Skill level and difficulty.** Riding a motorcycle requires more skills than driving a car. Unskilled riders account for a disproportionate number of motorcycle accidents. In 2001 in UK, more than one quarter of all motorcyclists killed in crashes did not have a proper motorcycle license.
- **High-risk behaviour.** Lighter and more powerful motorcycles such as sport and supersport bikes can encourage speeding, fast accelerating, and other high-risk behavior.

3. WORLDWIDE MOTORCYCLE USE AND ACCIDENTS

Motorcycles are becoming very popular due to some notable factors like great maneuverability, accessibility, ease of operation (Grava, 2003); despite the risk of motorcycle riding is extremely high. Motorcycles as one of the most affordable motorized vehicle in many parts of the world and, for a major portion of the world's population especially in Asia and some African region, they are also the most common type of motor vehicle. According to WHO (2013), 455 million motorcycles are in use worldwide in 2010 which is about 69 motorcycles per 1,000 people whereas around 782 million cars which means 118 per 1000 people. Statistics shows that in the period (2002-2010) the rate of increase per 1000 population in motorcycles (from 33 in 2002 to 69 in 2010) seems to surpass the rate of car growth (from 91 in 2002 to 118 in 2010) (Nguyen, 2013).

The increase of motorcycles in Asia especially in the last few decades has led to more motorcycle deaths as revealed by latest WHO Global Status Report. The Most of the world's motorcycles are occupied by Asians (79%), which is the most populated continent, still the rate of motorcycle fatalities is very high (78%) compared to other continents as shown in Figure 1.

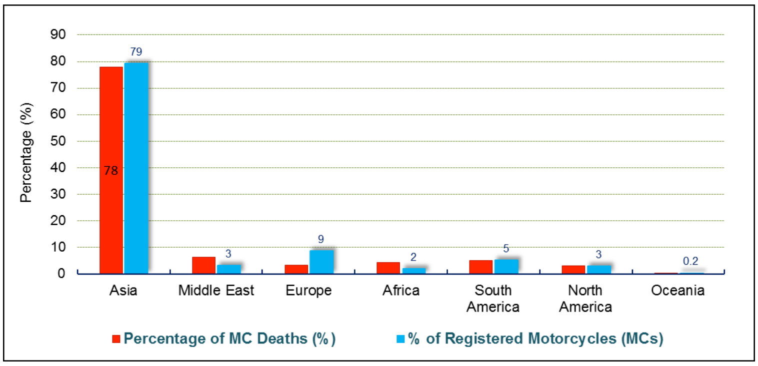

Figure 1: Percentage of motorcycle use and fatalities by continents according to WHO (2013)

Asia accounted for 80% of the world's motorcycle fleet of 290 million units and 90% of world motorcycle sales according to estimation in 2005. China accounts for 50% of yearly motorcycle production in Asia (17 million units) followed by India (8 million), Indonesia (5 million) and Vietnam (2 million). In India, motorized two wheelers comprised over 71% of registered motorized vehicles as of 2004 and continued to represent over 70% of annual vehicle sales through 2007. In Bangladesh, motorcycles occupy about 55% of all registered vehicles and registered ownership of motorcycles is increasing at an astonishing rate - more than 590% in the period between 1998 and 2012. Figure 2 shows the top ten countries in Asia which have greatest percentage of motorcycles per thousand people. Vietnam and Malaysia are two countries in Asia have more than one motorcycle for every three people. Indonesia and Thailand have approximately one motorcycle for every four people.

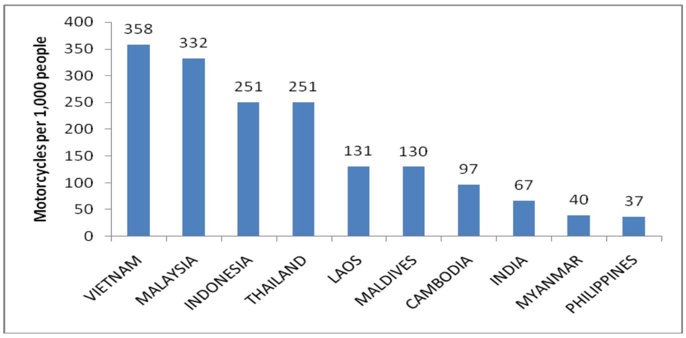

Figure 2: The 10 Asian countries with greatest number of motorcycles per 1,000 people (WHO, 2013)

Bangladesh has one motorcycle of around every twenty people compared to Vietnam and Malaysia have more than one motorcycle for every three people and Indonesia and Thailand have approximately one motorcycle for every four people. But the fatality rate in terms of number of motorcycles is very concerning than those countries with very high motorcycle occupancy rate. In fact, Bangladesh has one of the highest motorcycle fatality rates in terms of number of motorcycles in Asia - 34 motorcyclist deaths per 1,00,000 motorcycles, where in Indonesia, 25 motorcyclist deaths per 1,00,000 motorcycles.

In 2010, 23% of global road deaths were among motorcyclists costing enormously to the society. For example, fatal and serious crashes involving motorcyclists cost £1.8 billion annually in the UK alone. The annual cost of motorcycle accidents in USA is $16 billion as reported Government Accountability Office of USA in 2010. Malaysia lost at least $1.5 billion from motorcycle fatalities in 2010 only (Ghani et al, 2001). It is cogent that the huge economic loss incurred by motorcycle accidents every year can be minimized by investing in improvement in motorcycle facilities, where there is clear indication of motorcycles and subsequent crashes to increase.

4. KEY FACTORS OF MOTORCYCLE ACCIDENTS AND ROAD INFRASTRUCTURE HAZARDS

4.1 Excessive Speed and Motorcycle Accident

Motorcycles' exposure to road hazards and other vehicles have made them more vulnerable to accidents and increased the severity of accidents if occur. Since the mass of motorcycle is lower than car, normally the imapct will be higher if collision occurs. Therefore excessive speeding has been identified as one of the core important reasons for motorcycle accidents. An analysis of crashes at intersections in France involving a motorcyclist and another road user indicated that the initial speeds of motorcyclists involved in "looked-but-failed-to-see" crashes are significantly higher than in other types of crashes at intersections (Clabaux et al, 2011). Indeed speed management holds great importance in terms of reducing motorcycle involved accidents.

4.2 Road Infrastructure and Motorcycle Accidents

Studies related to accident analysis and contributions of the road infrastructure recognized that the variables like road geometry (vertical and horizontal alignment, width, cross slope, etc.), road sides (trees, fixed objects, poles, embankments etc.) and road surface (roughness, wheel ruts) can pose a unique risk to motorcyclists. Motorcycle stability can be affected more by changes in the shape, texture, or the skid resistibility of the road surface than other vehicles. Aspects of the environment and road surfaces such as potholes, loose gravel and impaired sightlines can be more hazardous for motorcyclists than for other road users.

Other road features that can be hazardous to motorcycles include insufficient shoulder and clear zone widths, rumble and median strips, road furniture placement and road marking visibility. Analysis of single vehicle crashes in New South Wales (NSW) during 2001-2005 found that road surface hazards were a contributing factor for 27% of motorcycle crashes on curves. Roadside objects pose a high risk for motorcyclists when crashes involve departing the roadway. The greatest proportion of motorcyclists killed in NSW was in motorcycle-object crashes (35 per cent of all motorcyclist fatalities). Trees and other types of vegetation are the most frequent objects hit, followed by roadside barriers, kerbs/gutters, then utility poles, median strips and walls. Evidence suggests that the risk of motorcycle accidents is increased by a number of factors that relate to the road environment, including (see The Motorcycle Council, NSW Road Safety Website):

- Adverse camber/ crossfall
- Blind corner
- Lack of appropriate warning
- Glare from poorly sited street lights
- Poor sight lines
- Speed hump
- Poor delineation of vehicle path
- Barrier kerbing
- Low kerbing
- Cobble stones or pavers
- Landscaping
- Traffic islands that are not lit or delineated for night visibility
- Culverts
- Uncovered drainage pits.

While the risk factors are based mainly on Australia study, many of highways are expected to apply in general in other settings including Bangladesh. Observational studies of accidents in the context of Bangladesh have documented may varied factors. Typically the principal contributory factors of accidents prevalent in urban areas are (Binnie partners 1994; Haworth 1995; Hoque 2006):

- Mix traffic with a variety of vehicle characteristics and speeds
- Failure to obey mandatory traffic regulations
- Conflicting use of roads
- Illegal and inconsiderate driving practices
- Adverse roadway and roadside environment
- Pedestrian and vehicle conflicts
- Failure to enforce traffic safety laws
- Inadequacy of police inspection and sanctions
- Inadequate and unsatisfactory of education of road users
- Poor detailed design of junctions and road sections
- Failure to give way and non-compliance of traffic rules
- Lack of lane disciplines
- Non-wearing of motorcycle helmets
- Failure to slow down approaching intersections
- Excessive speeding, overloading, dangerous overtaking
- Hazardous front ends of motor vehicles

5. ROAD INFRASTRUCTURE ASSESSMENT FOR MOTORCYCLE SAFETY

5.1 Motorcyclist Safety Assessment Results

The International Road Assessment Program (iRAP) - Bangladesh Pilot Project (iRAP 2010) which provided the first comprehensive infrastructure risk assessment of the N2 and N3 highways showed that hazards and deficiencies associated with road infrastructure and roadside environment are major contributors to motorcycle accidents. The safety ratings of these two major highways are mostly (92%) 2-star or less (out of possible 5-star) for motorcyclists - indicative of serious road infrastructure and environmental deficiencies. Further assessment of around 1400 kilometres of highways revealed that 81% of highway sections are 2-star or less for motorcyclists. The results of both assessments are presented in Table 1.

Table 1: Infrastructure Risk Assessment Results in terms of Star Ratings for Motorcycles

Star Ratings	**N2 and N3 Highways (2010)**		**1372 km National Highways (2013)**	
	Length (km)	Percentage	Length	Percentage
5-Star	1	0%	0	0%
4-Star	17	5%	3	0%
3-Star	8	3%	262	19%
2-Star	194	61%	295	22%
1-Star	99	31%	810	59%
Not rated	0	0%	2	0%
Total	**319**	**100%**	**1372**	**100%**

5.2 Typical High Risk Section for Motorcyclists

Figure 3: Typical High Risk sections for Motorcyclists: 1-Star Section on R880 Highway (left), 2-Star Section on N5 Highway (right)

Figure 3 illustrates a typical rural roadway sections which has poor star rating (1-Star and 2-Star) for motorcyclists indicating a very hazardous condition for motorcycle safety. The characteristics that led to poor motorcycle rating observed in those particular sections of highways are briefly discussed below:

- There are a significant number of illegal parking along the carriageway within the bazaar and near the junction thus resulting in a congested road network that creates various hazards for vehicles travelling.
- The presence of street vendors and parked vehicles restricts the use of the shoulder drop off.
- There are a number of advertising and commercial signs placed along the approach to the bazaar makes the road signs inconspicuous.
- There is no dedicated sidewalk or cross walk facilities for pedestrians and absence of regulatory/warning signs or signals.
- High speed, high occupancy through traffic mostly commercial vehicles very often have conflicts with local low speed operated minibuses, tempos and other non-standard vehicles, particularly NMVs.
- The volume of motorcycle traffic is very high and most of the riders don't use safety helmet which makes them one of the prime vulnerable road user groups.
- Uncontrolled frequent access and endless linear settlements create hazards along the highways.

6. SAFETY IMPROVEMENT OPTIONS

6.1 Road Infrastructure Improvement

The most effective countermeasure for motorcycle safety could be separation of motorcycles from other vehicles by “Segregated Path” or “Dedicated Lane” for motorcycles and these facilities have been successfully implemented in countries like Malaysia, Indonesia, Taiwan, Philippines, UK etc. iRAP has identified that the risk factor of zero associated with a motorcycle path with barrier assumes no opportunity for a vehicle motorcycle collision on that path. Without a barrier, it is there is a 10% increase in risk, assuming a rear-end or sideswipe type crashes of a vehicle on a motorcycle. Provision of a dedicated motorcycle lane on the roadway halves the risk of a motorcyclist crash compared with there being no provision at all and a segregated lane without a barrier is assumed to have one tenth of the risk of a dedicated motorcycle lane. According to iRAP, some of the major safety improvement options for motorcycles have been presented in Table 2.

Table 2: Motorcycle safety improvement options, costs and casualty reduction (Source: toolkit.irap.org)

Improvement options	Estimated Costs	Casualty Reduction
Roadside Safety - Hazard Removal	Low to medium	25-40%
Motorcycle Lanes	Medium	25-40%
Speed Reducing Treatments	Medium	25-40%
Road Surface Upgrades	Medium	25-40%
Parking Improvements	Low to medium	10-25%

6.2 Behavioural and Strategic Measures

Bangladesh needs to address strategic measures in the plan of action of improving motorcyclist safety. The idea of behavioural measures usually include helmet wearing, lane maintaining, speed controlling, waiting restriction in the vicinity of road, improved visibility, the use of daytime running lights etc. which can be implemented by proper enforcement, educational programs and publicity campaign. The applicable strategic measures could include the following key issues to focus (as identified by New South Wales Motorcycle Safety Strategy):

- The behaviour of both riders and pillion passenger who lack consideration for their own safety or that of other road users.
- The lack of courtesy and tolerance between motorcyclists and other road users.
- Riders need to better understand and manage road hazard risks.
- The issue of unlicensed and reckless riding.
- The need to continuously monitor and improve pre and post license rider training.
- The need for more effective distribution of safety information to riders.

7. CONCLUDING COMMENTS

Motorcycle safety is a vital concern deserving immediate attention. A number of factors influencing motorcycles casualty risks are discussed. The safety ratings of the major highways are mostly (81%) 2-star or less for motorcyclists. Engineering safety on roads is clearly a priority issue and motorcycle accidents cannot be prevented until safety treatments are built on the road infrastructure. Some effective road infrastructure engineering, behavioural and enforcement measures are briefly highlighted in the paper. Greater understanding of the underlying factors associated with motorcyclists' risk is a critical step in developing strategies, policies and effective measures and thereby making motorcycling a more viable and safe mode of transportation in Bangladesh. Importantly, there is a need to need to develop and design measures adapted to local context which require extensive studies and research.

ACKNOWLEDGEMENTS

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REFERENCES

Bairgi, B. K. (2012), 'Assessment of Motorcycle and Bicycle Safety on Dhaka-Aricha Highway using iRAP tools', B.Sc. Thesis, Department of Civil Engineering, Bangladesh University of Engineering and Technology

Binnie partners (1994), " Preparation of Structure plan, Master plan and Detailed Area plan for Chittagong, traffic and transportation in Chittagong, Component 2C: present situation report, traffic and transportation in Chittagong, Project BGD/88/052, UNDP/UNCHS (Habitat), Govt. of Bangladesh

Clabaux, N., Brenac, T., Perin C., Magnin, J., Canu, B. and Elslande, P. V. (2011), 'Motorcyclists' speed and "looked-but-failed-to-see" accidents', Accident Analysis and Prevention, France, 2012 (49), pp 73-77

Ghani, N., Faudzi. M., Umar., Radin (2001), "The Value of Life and Accident Costing: a Willingness to Pay Study amongst Young Motorcyclists in Malaysia', Joumal of the Eastem Asia Society for Transporration Studies. Vol.4. No.5. October. 2001

Grava, S., (2003), 'Urban Transportation Systems: Choices for communities', McGraw-Hill, US

Haworth, N. (1995), "Road User Behaviour and Safety in a Developing Country" Proceeding of the 7th World Conference on Transport Research, Sydney, July 1995

Hoque, M. M. (2006). Road Safety in Bangladesh: The Contemporary Issues and Priorities. Proceedings published on International Conference on Road Safety in Developing Countries, BUET, Dhaka, Bangladesh

Hurt, H. H., Jr., Ouellet, J. V., Thom, D. R., (1981), 'Motorcycle Accident Cause Factors and Identification of Countermeasures', Final Report, DOT-HS-F-01160

iRAP (2010 & 2013), International Road Assessment Program Bangladesh Technical Report

NSW Motorcycle Safety Strategy 2012 – 2021, TfNSW 12.052ISBN 978-1-922041-44-9 CAT No. 45094052

Nguyen, H. H. (2013), 'A Comprehensive Review Of Motorcycle Safety Situation in Asian Countries', Journal of Society for Transportation and Traffic Studies (JSTS) Vol.4 No.3

toolkit.irap.org, A Website with Free Information on the Causes and Prevention of Road Crashes

Vulnerable Road Users in the Asian and Pacific Region, Asian Development Bank

World Health Organization (2013), Global Status Report on Road Safety

www.nolo.com, Motorcycle Accidents: An Overview

www.streetdirectory.com, Motorcycles More Dangerous Than Cars

CONTENTS (Geotechnical Engineering)

Proceedings of the 2nd International Conference on Civil Engineering for Sustainable Development (ICCESD-2014), 14~16 February 2014, KUET, Khulna, Bangladesh, ISBN: 978-984-33-6373-2 (CD-ROM)

STUDY ON SOME PHYSICAL AND STRENGTH CHARACTERISTICS OF CEMENT STABILIZED SOIL

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ABSTRACT

This study is mainly concerned with the laboratory experiments of strength of cement stabilized soil at different cement contents with different water contents required as well. Firstly, soil samples were collected from a site near to KHAN JAHAN ALI HALL (KUET), Khulna. The tests of Liquid limits, Plastic Limit, Plasticity index, Specific gravity, Grain size distribution were performed. Initial LL, PL, PI, Specific gravity are found to be 55.78%, 28.2%, 27.58% & 2.71 respectively. After conducting the grain size analysis it was found that the collected soil was Silty clay. Then to stabilize the soil a popular cement brand (king Brand) was used. By mixing the cement at different percentage (5%, 10%, 15%, 20% & 25%) the unconfined compressive strength test were performed conventionally in the laboratory. This test includes pasting, molding, curing and finally the compression tests. The strength was measured for the period of 1, 3, 7, 14 & 28 days. After conducting the test, the failed samples were collected for further tests. It was observed that with the increase of cement LL decreases while the PL and specific gravity increase. Finally some graphical relations were established between various physical properties like LL, PL, specific gravity, PI with the mixing cement or water content.

Keywords: *Title, abstract, objective, results, discussions, conclusions*

1. INTRODUCTION

Soils are primary media for construction work and it's bearing capacity is generally governed by its strength and compressibility characteristics, the supporting power of loose or soft soil lying close to the ground surface is very low which always leads to use uneconomical foundation for construction work. Therefore, to improve the foundation soils, mixing admixtures such as cement, lime and fly ash can be an option. Kazi Fattaur Rahman (2002), worked on the change of different percentage of water on collected soil samples. But in this project work emphasize was given on the effect of different percentage of cement on collected soil samples and the interrelation amongst different physical characteristics. Strength and compressibility characteristics of stabilized soil are studied in the laboratory by mixing a better stabilizing agent like cement at a percentage of 5%, 10%, 15%, 20% & 25% etcetera. and it was found suitable for the improvement of the subsoil condition in this region for especially the construction of shallow foundation for buildings and sub grades for highways.

Rapid urbanization followed by population growth requires various types of civil engineering infrastructures and facility services. Because of scarcity of suitable lands it has become difficult to select an appropriate location for construction. The negative effects of soft ground like low strength, stability, high settlement, differential settlement or time-settlement sometimes offer no choice other than rejection-replacement, excavation and dumping as a waste. Because of the environmental consideration dumping of rejected soft soil also increases cost effectiveness of any construction project. Growing environmental awareness about contaminated soil because of human or geological activities has also become a burning issue. Consequent study shows that the construction regarding a method pointing the expiration puts emerging effect on the base line, that is why the stabilization technique came forward. The definition of cement content can be referred to as the ratio of the weight of cement being mixed to dry weight of soil. However, the engineering behavior of cement- admixed clay is also affected by clay water content present in the clay-cement paste. Consequently, a new parameter, called total water content to cement content ratio, was proposed, stressing that such a parameter is a prime factor governing the engineering behavior of cement –admixed clay.

2. OBJECTIVES OF THE STUDY

a) To observe the effect of cement content on properties of stabilized soil.
b) To observe the influence of initial water content on cement stabilized soil.
c) To find out the relationship between the strength and deformation characteristics.
d) To find out the optimum amount of admixture content for stabilization of soil.
e) To find out the variation between strength and deformation characteristics with various proportions of admixture contents for the stabilized soil.

3. METHODOLOGY

Steps to be followed in this project work:

Firstly, soil samples were collected from a site near to KHAN JAHAN ALI HALL (KUET), Khulna. The initial water content of the collected soil samples has been determined. The tests for Liquid limit, Plastic Limit, Plasticity index, Specific gravity, Grain size distribution were performed. To ensure that the soil was not organic, the organic content of the collected soil samples has been determined. Then to stabilize the soil a popular cement brand (king Brand) was used. By mixing the cement at different percentage (5%, 10%, 15%, 20% & 25%) the unconfined compressive strength test were performed conventionally in the laboratory. This test includes pasting, molding, curing and finally the compression tests. Unconfined compressive strength was measured for the period of 1, 3, 7, 14 & 28 days. After conducting the test, the failed samples were collected for further tests for the determination of Liquid limit, Plastic limit, Plasticity index, Specific Gravity etc.

3.1 Collection of soil samples

Soil samples are collected from a depth few feet below the existing ground surface from a site near to Khan Jahan Hall of KUET, Khulna.

3.2 Physical and mechanical properties of cement

The physical and mechanical properties of cement have been collected from the engineering materials laboratory of Civil engineering department, which has been given below:

a) Type of cement: King Brand
b) Normal consistency: 26%
c) Initial setting time: 50 min
d) Final setting time: 5 hrs. 45 min
e) Fineness: 3.86%
f) Compressive strength: 3 days = 1750 psi, 7 days =2900 psi

3.3 Preparation of soil cement samples

Following steps were followed for preparation of soil cement stabilized mold:

Step-1: Taking the required samples and cement.

Step-2: Mixed the soil and cement at a container, adding necessary and sufficient water to make a paste.

Step-3: putting the paste into the mold very hardly and carefully by the fingers so that no air void can be entrapped in the sample.

Step-4: Ejecting the samples from the mold and then putting them in air exposure for a while and keeping under water for curing.

Step-5: After that, the specimens were wrapped by air tight polythene and put it under water for curing till the designated period.

3.4 Flow chart

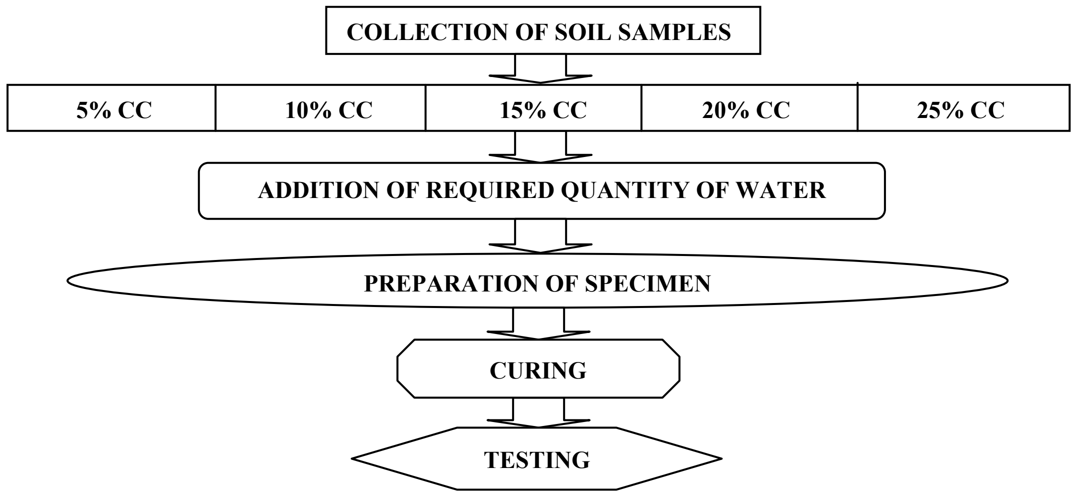

Figure 1: Flow chart of the methodology

4. RESULTS AND DISCUSSIONS

4.1 Physical properties of soil samples used in the study

Table 1: Physical properties of collected soil samples

Initial water content (%)	Organic content (%)	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	Specific gravity	Type of soil
37.49	4.8	55.78	28.2	27.58	2.71	**Inorganic clay with high plasticity**

Table2: Determination of LL

Can no.	1	2	3	4
No of blows, N	15	19	35	26
Mass of can(gm)	22.6	22.3	22.1	22.4
Mass of can +wet soil	53.2	56.7	57.1	54.7
Mass of can +dry soil	41.7	44.2	44.9	43.1
Mass of dry soil	19.1	21.9	22.8	20.7
Mass of water	11.5	12.5	12.2	11.6
Water content, W%	60.20942	57.07763	53.50877	56.03865
Liquid limit	**55.78**			

Graphical presentation of LL:

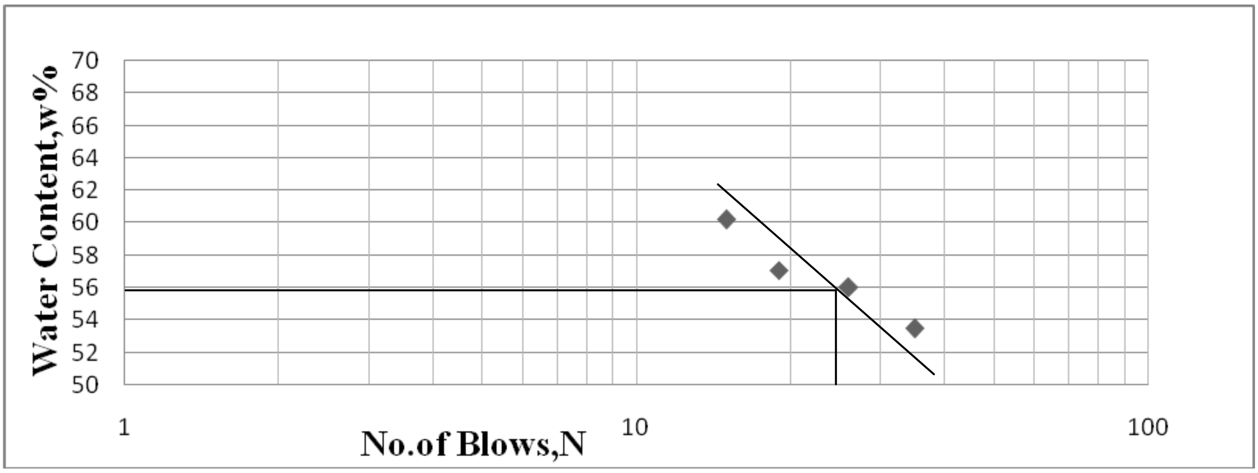

Figure 2: Liquid limit curve

4.2 Grain size distribution

This test is performed to determine the percentage of different grain sizes contained within a soil. The mechanical or sieve analysis is performed to determine the distribution of the coarser, larger-sized particles, and the hydrometer method is used to determine the distribution of the finer particles.
Standard Reference as ASTM D 422 - Standard Test Method for Particle-Size Analysis of Soils:

1) Gravel: 76.2>4.75mm
2) Sand: <4.75 to 0.075mm
3) Silt: <0.075 to 0.005mm
4) Clay: <0.005 to 0.001mm

Table 3: Grain size analysis-Hydrometer method

Prtcl. Size, Dmm	0.037	0.027	0.02	0.0125	0.009	0.007	0.0037	0.0035	0.003	0.002	0.0022	0.0013
% Adstd. Finer PA	89.5	85.64	81.31	70.25	62.55	54.85	47.15	35.60	31.76	27.91	24.1	16.1

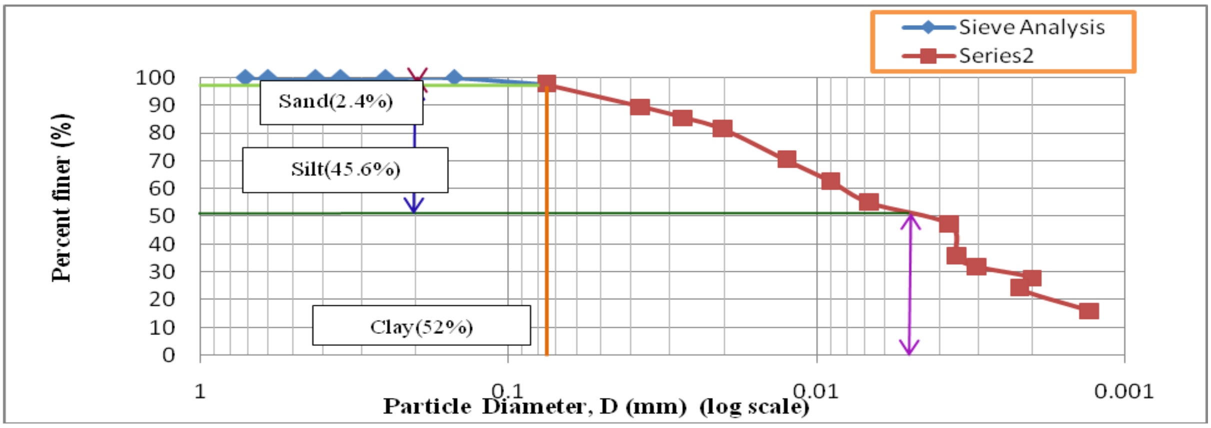

Figure 3: Grain size distributions of collected soil particles

From the grain size distribution curve, by plotting the values of particle size against the adjusted percent finer (%) it has been found that soil collected from the locality is Silty clay (clay- 52% & silt- 45.6%).

4.3 Influences of mixing cement content on the strength

Table 4: Change of compressive strength with curing periods

CC%	Periods(days)					
	1	3	7	14	28	
5	150.1	216.3	181.3	180.3	198.4	Stress (kPa)
10	210.6	327.9	404.9	552.9	603.8	
15	669.5	772.5	948.3	723.2	1082.6	
20	665.2	762.53	831.7	1088.4	1783.9	
25	795.23	1191.7	1300.5	1518.9	1866.7	

Some of the graphical representations of stress vs. strain have been deliberately shown in plain graph paper:

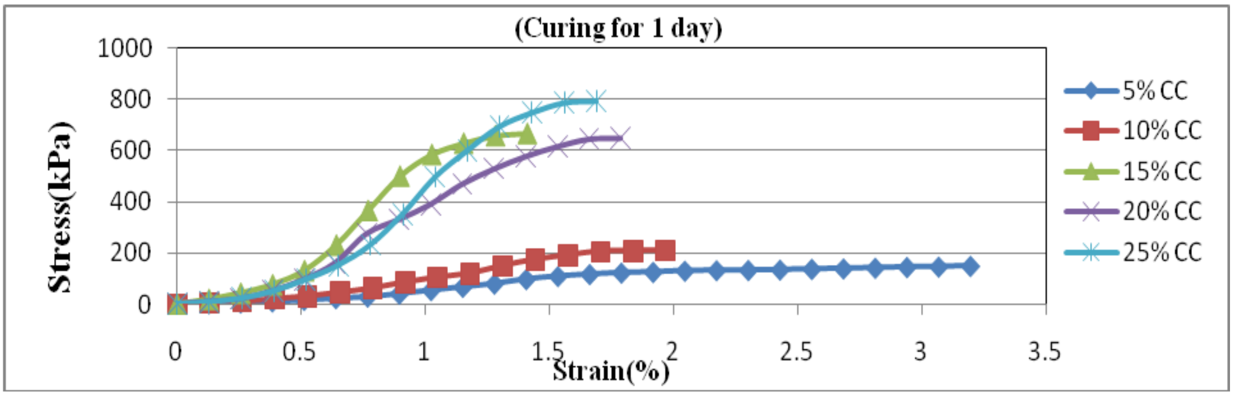

Figure 4: Typical stress-strain behavior of cement stabilized soil for specimen-1(For curing period of 1 day)

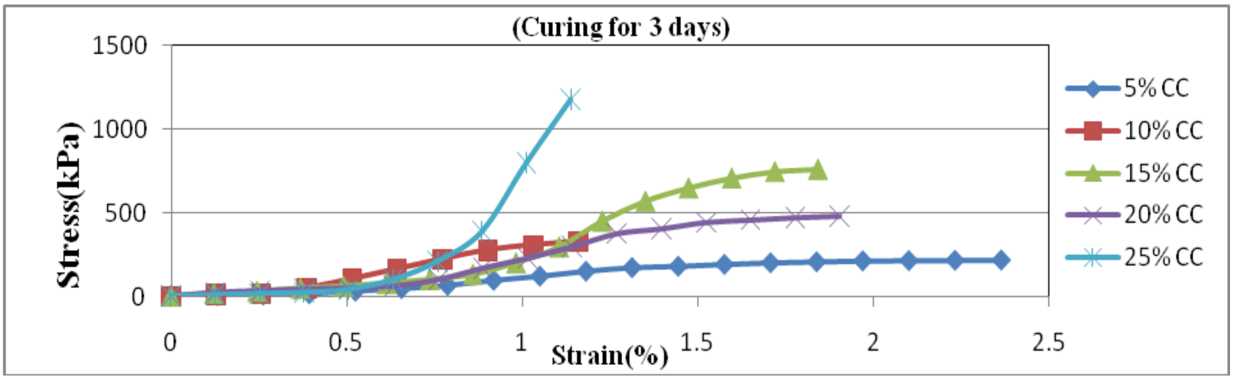

Figure 5: Typical stress-strain behavior of cement stabilized soil for specimen-2 (For curing period of 3 days)

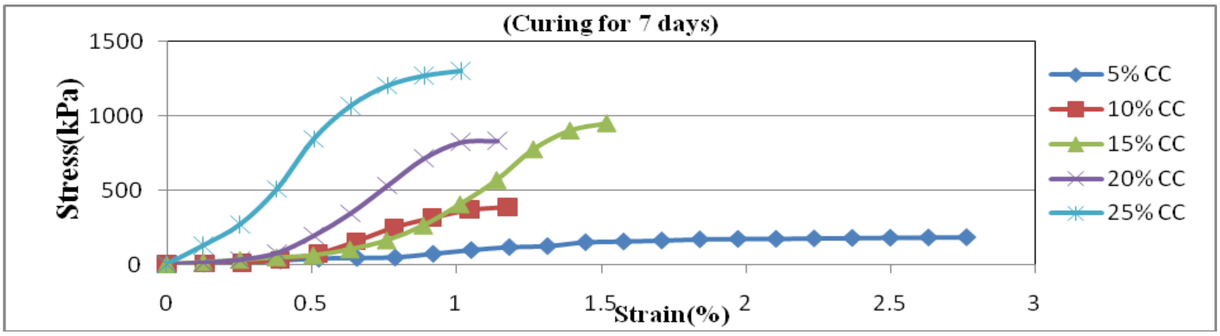

Figure 6: Typical stress-strain behavior of cement stabilized soil for specimen-1 (For curing period of 7 days)

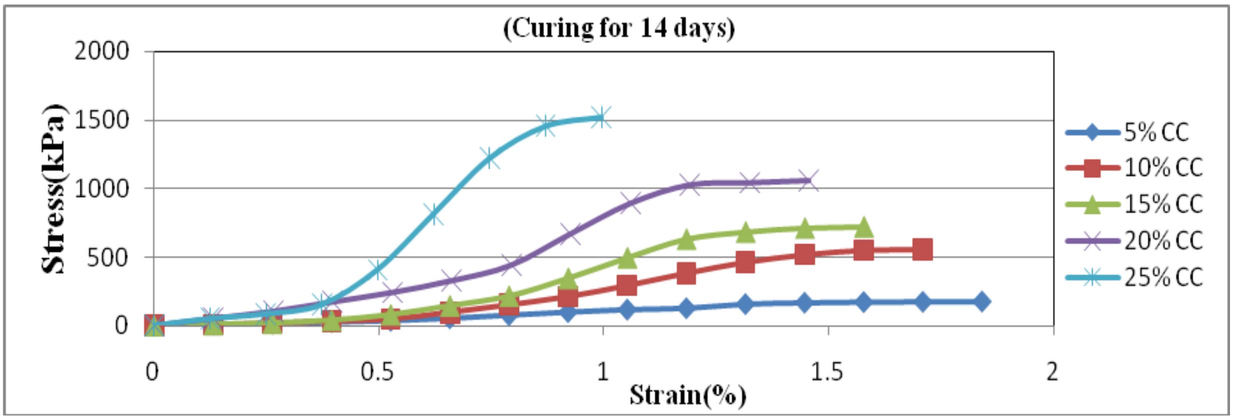

Figure 7: Typical stress-strain behavior of cement stabilized soil for specimen-2 (For curing period of 14 days)

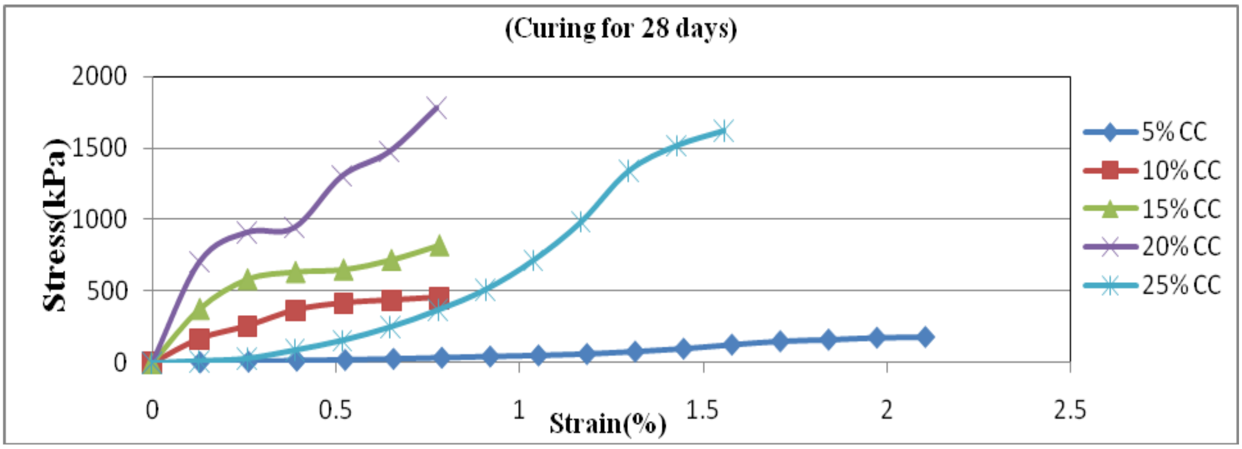

Figure 8: Typical stress-strain behavior of cement stabilized soil for specimen-1 (For curing period of 28 days)

4.4 Change of compressive strength with curing periods and cement percentage

It is found that the strength varies with the curing periods. But for all the curing periods it does not increase uniformly though the curing periods are increasing. So the strength is increased for a particular curing period and with the increment of curing periods, it does not increase uniformly as it is expected.

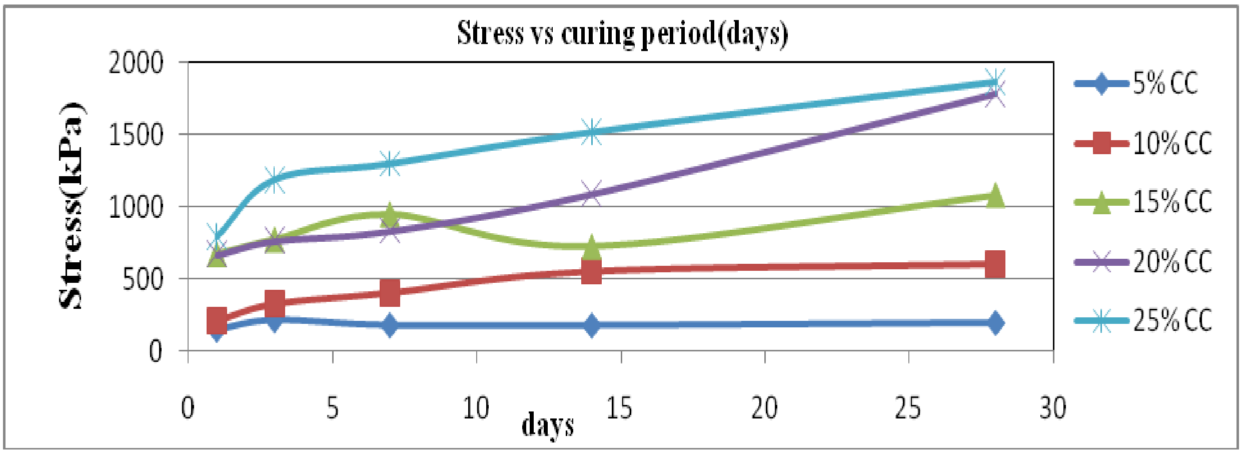

Figure 9: Unconfined compressive strength vs. curing period

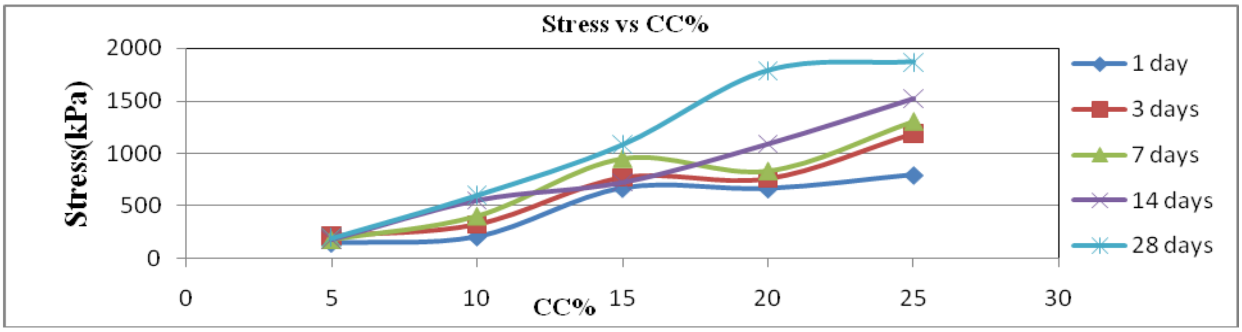

Figure 10: Unconfined compressive strength vs. mixing cement content

4.5 4.5 Interrelationships of physical properties

With the increase of the cement content from 5% to 25%, a significant change in physical properties is observed in cement stabilized soil. The liquid limit of the soil samples decreases from 46.44 to 42.65, Plastic limit increases from 28.81 to 30.32 and the specific gravity increases from 2.718 to 2.741. It is also explainable that the above properties also vary from the original collected soil samples.

Table 5: Physical properties of cement stabilized soil

Sample no.	CC%	WC% (used)	Liquid limit (%)	Plastic limit (%)	Plasticity index	Specific gravity
1	5	33.96	46.44	28.81	17.63	2.718
2	10	34.25	45.65	28.93	16.72	2.723
3	15	35.15	44.94	29.27	15.67	2.73
4	20	36.79	43.25	29.74	13.51	2.738
5	25	38.55	42.65	30.32	12.33	2.741

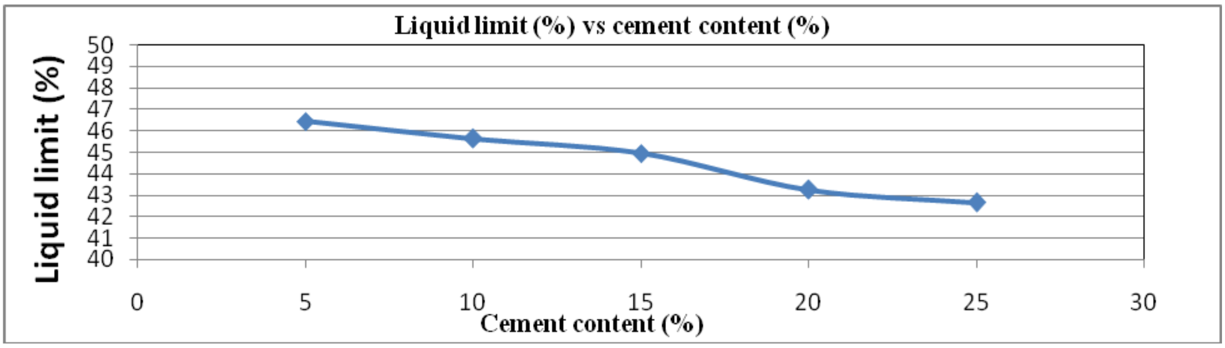

Figure 11: Variation of liquid limit with added cement content

The figure given above shows the decrease of liquid limit with the increase of cement content in stabilized soil samples is almost linear. On the other hand, plastic limit of soil increases onto the increase of cement content which has been given below.

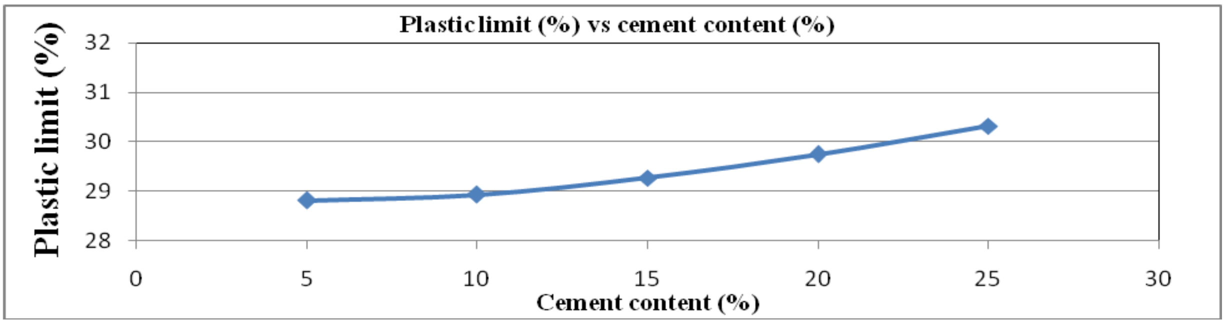

Figure 12: Variation of plastic limit with added cement content

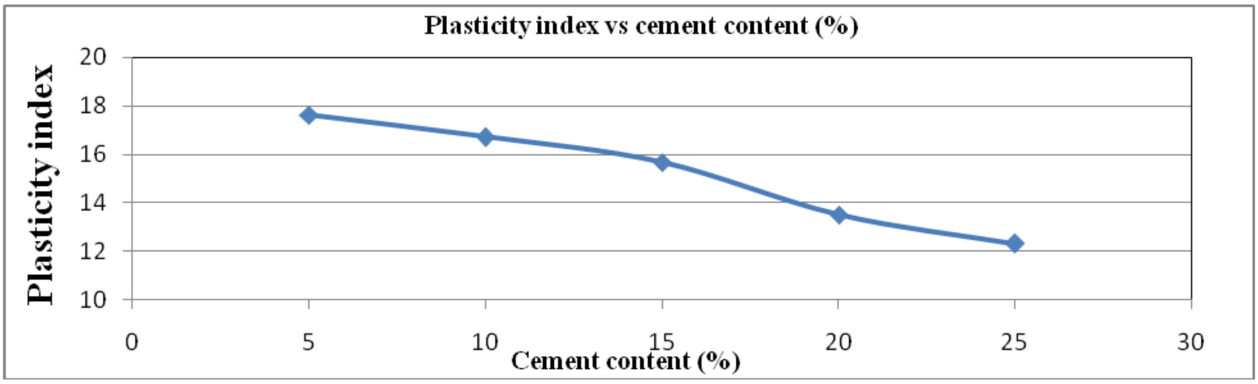

Figure 13: Variation of plasticity index with added cement content

It can be satisfied that the relation of Plasticity index with CC% is almost linear.

The specific gravity of the cement stabilized soil increases with the addition of the cement content, because the void spaces between the soil particles are filled up by the cement particles that have been shown with the graphical representation.

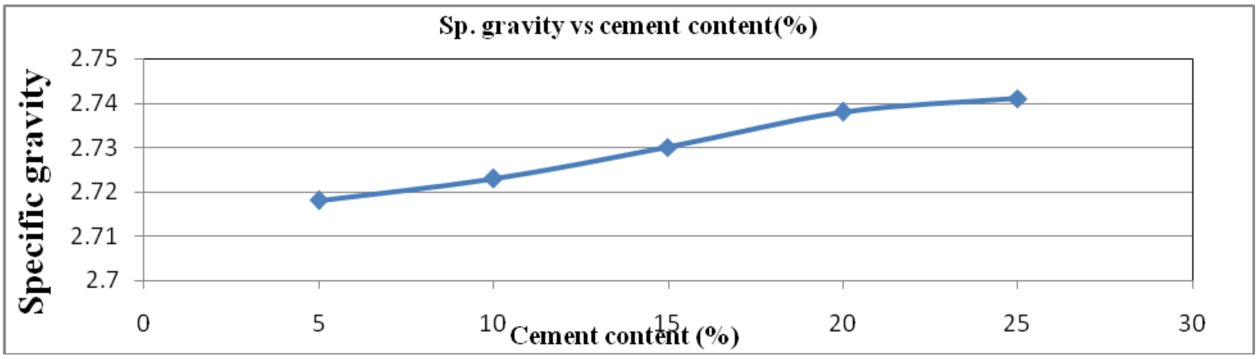

Figure 14: Variation of specific gravity with added cement content

5. CONCLUSIONS

Based on the test results present the following conclusions could be drawn:

The physical properties of soils mixed with different percentage of cement change with cement content. Strength of the soft soil collected can be increased significantly upon the addition of the Portland cement. The strength of the soil increases with curing period. The bearing capacity of the soil increases upon the addition of cement. The liquid limit decreases with the increase of cement content. With the increase of cement content the plastic limit increases. Specific gravity of the prepared cement stabilized soil increases in comparison to the normal soil.

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REFERENCES

Das, B.M. (2002) "Principles of Geotechnical Engineering." (Fifth edition), Thomson Book House, Canada.
Kazi Fattaur Rahman (2002), 'Effect of Mixing Water Content On The properties Of Cement Stabilized Soil' (A project report submitted to the Department of Civil Engineering, KUET).
Bowels, J.E. 'Foundation Design Analysis & Design' (5th edition) McGraw- Hill, Singapore, 1997.
Braja M. Das. 'Principles of Geotechnical Engineering' (5th edition)
Editor: T. Willam Lambe, Soil Testing for Engineers from MIT.
Punmia, B.C. (1994). 'Soil Mechanics & Foundation' (13th Edition).
Wayne C. Teng, 'Foundation Design' (Eastern Economy Edition).

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STATISTICAL EVALUATION OF BEARING CAPACITY OF KHULNA SUB-SOIL

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ABSTRACT

For building of small height with one or two storeys, the owners are not interested for soil investigation. For these cases the foundation are designed on the basis of assumed average bearing capacity. The main objective of this research work is to determine an average bearing capacity for the design of low-rise building to be constructed in Khulna city corporation area. Some soil samples were collected from KUET (Khulna University of Engineering & Technology) campus and tested in the Soil Mechanics laboratory of KUET to determine the bearing capacity. Soil reports from more than sixty locations of Khulna were collected from CRTS (Consultancy, research and testing services) of Civil Engineering department KUET, Khulna. From these reports, the values of bearing capacity were taken for the purpose of analysis. Many equations are available in the determination of the bearing capacity of soil such as Terzaghi, Meyerhof and Hansen etcetera. Among these equations, the Terzaghi Bearing Capacity equation was used for the estimation of bearing capacity considering its simplicity and reasonable accuracy. Since the bearing capacity depends also on layer thickness and stratification of soil, soil profile of different locations were drawn. From the reports, the mean, median, mode, standard deviation and coefficient of variation of different values of bearing capacity were determined. The estimated average bearing capacity of Khulna sub-soil of at 5 ft and 10 ft depth are 55.31 kN/m2(0.523 tsf) and 47.097 kN/m2(0.447 tsf) respectively.

Keywords: *Bearing Capacity; Soil Investigation; Mean; Median; Mode; Standard Deviation; Coefficient of Variation.*

1. INTODUCTION

The concept about the subsoil parameters of any project site is very important to plan and design the foundation of the concerned structure, so that the structure after it construction would remain safe and stable. Khulna city is situated on the Southern region of Bangladesh. The alluvial deposits from different rivers form the soil in this region. Moreover, it was once a part of Sunderban, so the soil of this region is mostly soft and organic having low bearing capacity.

In addition, "Khulna city" is moving forward with large development project including construction of buildings, oil storage tanks, long span bridges, harbors, port structures, flood protection embankments, barrages etcetera. That is why, it is very important for foundation engineer to go for a detail investigation of sub-soil for a detailed knowledge and sound understanding of bearing capacity before going for construction of structures.
With this end in view, the authors have undertaken a comprehensive program on statistical analysis of bearing capacity of Khulna sub-soil with the following objectives:

a) To execute exploratory test in order to collect sufficient data for the safe and economic design of foundation of structure.
b) To draw the bar diagrams for bearing capacities of different location of Khulna city.
c) To draw soil profile of several location of Khulna City.
d) To construct histograms for the values of bearing capacities of different location of Khulna city.
e) To determine mean, mode, median, standard deviation and coefficient of variation of bearing capacities of different locations of Khulna region.
f) To determine an average bearing capacity of Khulna sub-soil.

1.1 Bearing Capacity

The conventional method of foundation design is based on the concept of bearing capacity or allowable bearing pressure of the soil. The bearing capacity is defined as the load or pressure developed under the foundation, without introducing damaging movements in the foundation and in the superstructure supported on the foundation. Since damaging movements may result from foundation failure (collapse) as well as from excessive settlement, the following criteria must always be satisfied:

a) Adequate factor of safety against failure (collapse)
b) Adequate margin against excessive settlements.

In order to provide an adequate factor of safety against foundation collapse, the ultimate bearing capacity must be known. Usually a factor of safety of 3 is used for maximum load normally expected to act upon the foundation (Das 2002).

Bearing capacity equations suggested by the several authors are given below:

Terzaghi (1943):

$q_{ult} = CN_C S_C + qN_q + 0.5\gamma B N\gamma S\gamma$

$N_q = a^2/a \cos^2(45+\phi/2)$

$A = e^{(0.75\pi-\phi/2)\tan\phi)}$

$N_c = (N_q-1)\cot \phi /2$

$N_\gamma = \tan \phi/2(k_{p\gamma}/\cos^2 -1)$

For:	strip	round	square
$S_c=$	1.0	1.3	1.3
$S_c=$	1.0	0.6	0.8

Where, C=cohesion of soil (kN/m^2),
γ=unit weight of soil (kN/m^3),
$q=\gamma D_f$, D_f=depth of footing (m),
B= width of footing (m),
N_C,N_q,$N\gamma$=bearing capacity factors,
S_C,$S\gamma$=shape factors.

Meyerhof (1963):

Vertical load: $q_{ult}=CN_c s_c d_c + q^{-} N_q s_q d_q + 0.5_\gamma B N_\gamma s_\gamma d_\gamma$

Inclined load: $q_{ult}= CN_c d_c i_c + 0.5_\gamma B N_\gamma d_\gamma i_\gamma$

Between these two equations, Terzaghi bearing capacity equation is widely used for the estimation of bearing capacity due to its simplicity and reasonable accuracy.

2. METHODOLOGY

2.1 Field and Laboratory Investigation

2.1.1 Collection of Soil Samples

In this investigation, soil samples were collected from the selected six sites of KUET campus and the physical properties and bearing capacity of soil were determined in the laboratory of KUET. Soil samples were taken at a depth of about 5 ft from the existing ground surface. The laboratory investigation made on the soil samples have been described in this chapter. To investigate the behavior of soil it would be very desirable to use the undisturbed sample.

2.1.2 Laboratory Testing

Various laboratory test were done to determine the physical properties of soil including

a) Atterberg Limits.
  - Liquid limit
  - Plastic limit

b) Wet and Dry Density.
c) Natural Moisture Content.
d) Unconfined Compression (UC) Test.

2.2 Data Collection and Storage

Sixty-six (66) sub-soil investigation reports containing two hundred nineteen (219) borehole data from CRTS for different places of Khulna zone were collected. All soil data were stored in a Microsoft Excel database. This Excel sheet consists of borehole data, depth, soil type in various depths and so on.

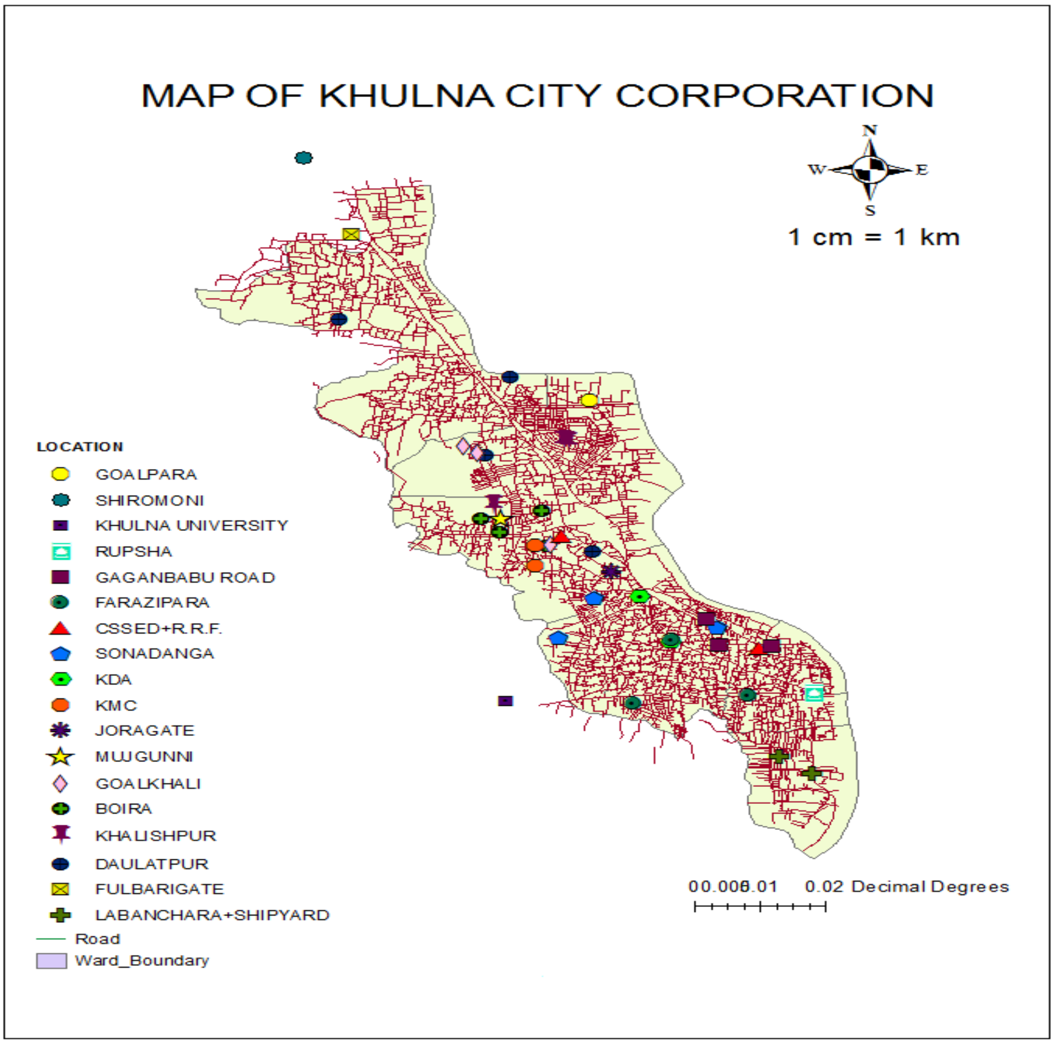

Figure-1: Borehole Location of Khulna Region

After this, the Khulna map formed the base layer for development of geographic information system (GIS) model (see Fig-1). The map had been developed in view of two aspects, first for locating the bore logs to the utmost accuracy on a scale of 1:100000 and second for identification of bore logs by various symbols. The digitized map had several layers of information. Some of the important layers considered were the boundaries (outer and administrative), highways, major roads, minor roads, streets, and borehole locations (Ansary, 2010).

2.3 Procedure

Soil samples from KUET campus were collected.

Collected samples were tested in laboratory of KUET to determine the physical properties and bearing capacity.

Sub-soil investigation reports from more than sixty sites of Khulna were collected.

Bar Diagrams of different locations of Khulna zone were drawn.

Soil profiles of different locations of Khulna zone were drawn.

From these reports histograms were constructed.

Mean, Mode, Median, Standard Deviation and Coefficient of Variation of the values of bearing capacity were calculated.

3. RESULTS AND DISCUSSION

This paper describes the results obtained from the laboratory tests conducted by standard methods. Moreover, it represents the bearing capacity of different locations of Khulna sub-soil, soil profile, histogram of bearing capacity and statistical parameters of the values of bearing capacity of soil.

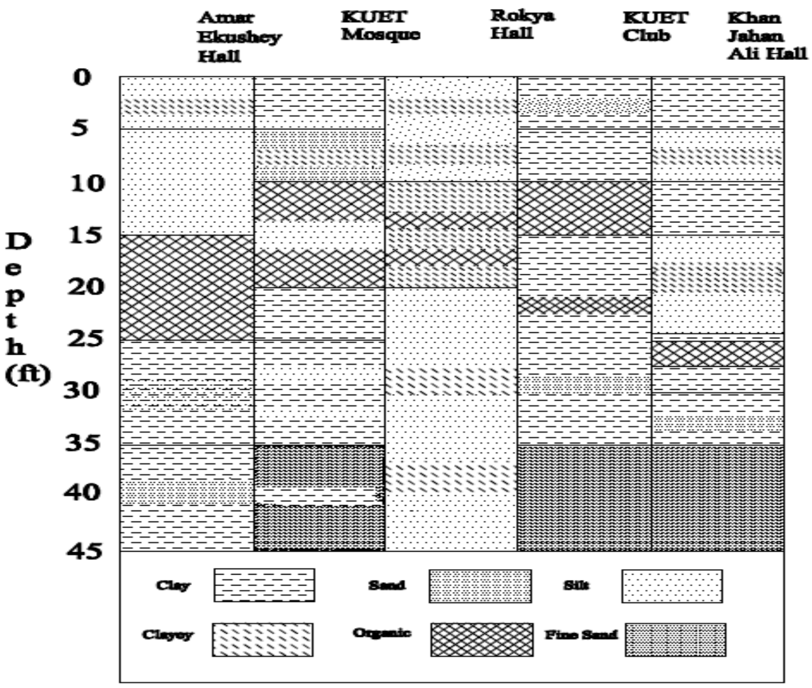

Figure-3: A Typical Soil Profile (KUET campus)

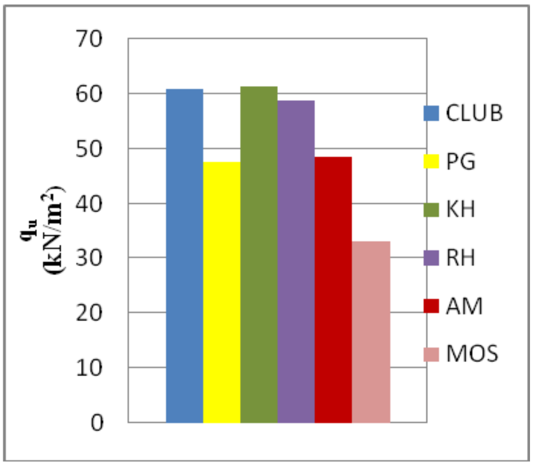

Figure-2: Histogram of bearing capacity of a typical site (KUET campus)

The Mentioned figures (fig-2 and fig-3) show the typical soil profile of Khulna region (KUET campus) and the histogram of bearing capacity of a typical site (KUET campus).

Table-1: Bearing capacity of Khulna City at a depth from 0-5ft

Location	Qu(kN/sqm) Average	Location	Qu(kN/sqm) Average
Boyra	40.5	Mujgunni	38.58
Khalishpur	34.5	Khulna Shisu Shadan (Girls)	72.33
Khulna 4 no gate	49	Jail khana Food store house	25.27
KMC	62	Fulbari Gate	48
Sheikh Para	57	Jora Gate	103.41
Sonadanga	57.5	KDA Plot no 122 Mujgunni	69.67
100 Bed Diabetic Hospital	30	RPACT	45.67
Khulna Thana Quarter	60	Police Phari, Chandmari, Rupsha	50.77
T.T.C	34	Central Mosque, KU	31.23
Goalkhali	35.25	Padma Oil Company, Daulatpur	64.30
Shisu Sadhan	66	Imam Training Center, Boyra	61.10
BOC	29.5	Regional Environment Research Center	63.33
Boikali	38	Residence of O.C., Sonadanga	47.98
Tarer Pukur	52	Residential Building, at 3 no. Baniakhamar Mouza	74.06
KMC (Mosque)	59	Residential Building, Kalishpur	30.32
KMC (Student Hall)	53	Hadi Tower, Gagan Babu Road, 2nd Lane	40.0
ICMA	52	Residential Building, Maheswar Pasa, Daulatpur	38
Fire Service	62	Six Storied Building,R.R.F. Police Line	53.55
New Market	48	Naval Colony	71.90
Labanchara	25.15	Shipyard School & College Building	44.81
Goalkhali(Hostel Building)	96.22	Divisional Server Station	63.09
Shiromoni	76.45	Goalpara	70.73
Delta Life Ins. Co. Ltd.	52	Farajipara	81.5
Moheswarpasha Shisu Sadhan	95.51	Plot no 389, Mujgunni R/A	31.9
Rupsha	67.41	Helatola	51
S.I & A.S.I	77.05	KMC(Gymnasium)	64

Senhate River Fire Station and Civil Defence, Daulatpur	95.37	Shisu Shadan (thumbi house)	70
Women Oppression Protection Project at Five Divisional Towns	80.5	Gallamari Bridge	65.1
Postal Training Center, Boyra	39.61	Ahashan Ahmmad Road	93.0
Sobujbag	64.90	B.N Collge	81.8
Zilla Police Line,Shiromoni	90.44	10 Storied Commercial Building G.M.Baksh &CO., Helatola	78.55
Khulna WASA	97.86	Tarer-Pukur (Residential Building)	79.39
Student Hostel (No 2) at KMC	80.03	Residential Building , FulbariGate	33

Table-1 represents the value of bearing capacity (in kN/m^2) of soil different locations of Khulna city at a depth from 0-5ft.

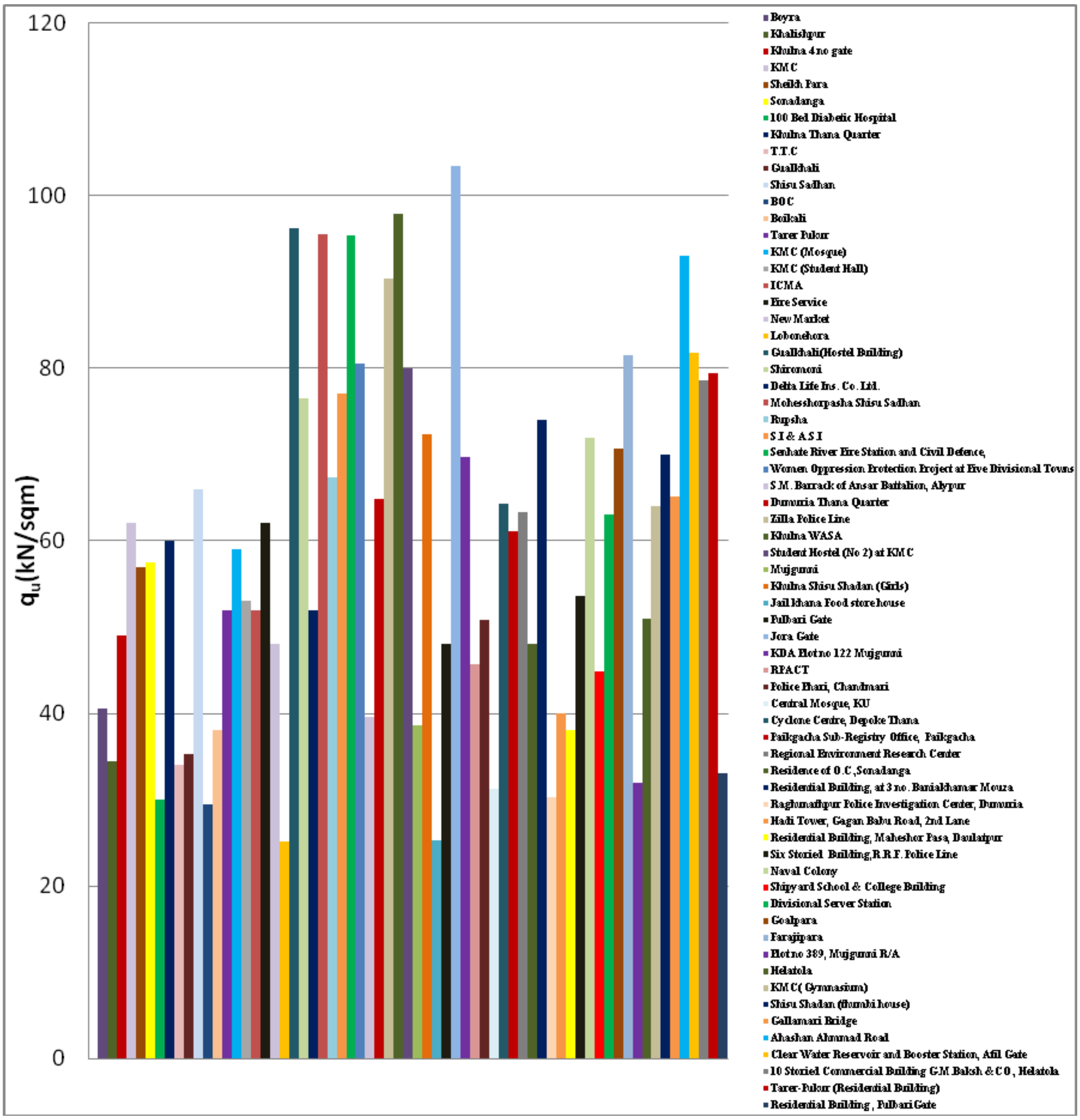

Figure-3: Histogram of average Bearing Capacity of Khulna sub-soil for at a depth from 0-5ft

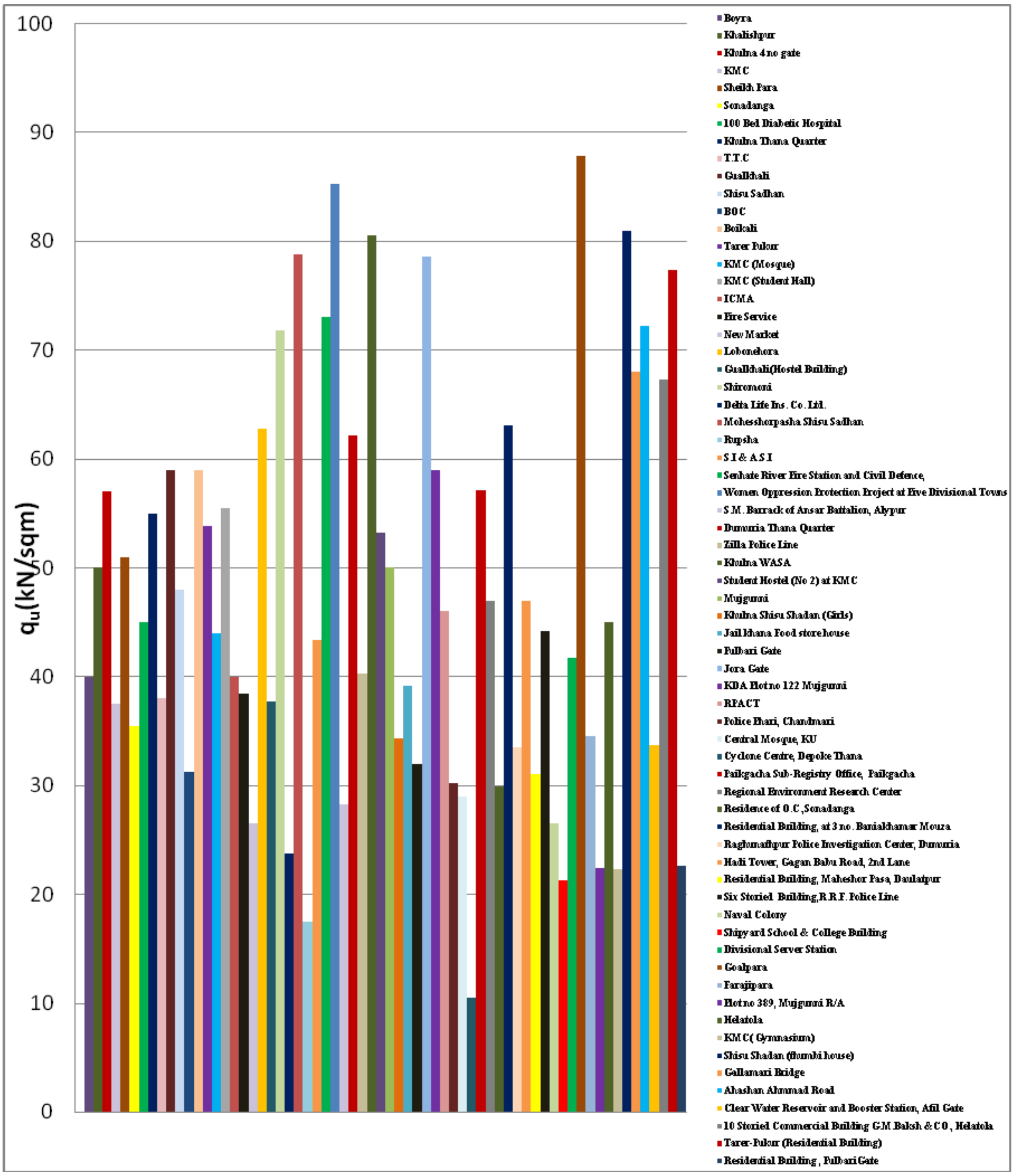

Figure-4: Histogram of average Bearing Capacity of Khulna sub-soil at a depth from 5-10 ft

Figure-3 as well as figure-4 illustrates the Histogram of different locations of Khulna zone at a depth from 0-5 ft and 5-10 ft respectively. In fig-3, the lowest average bearing capacity is found at Labanchara. In Joragate (fig-3), there is obtained maximum average bearing capacity. Again, in figure-4, the highest value is found at Goalpara whereas the lowest value is obtained in Daulatpur.

Table-3: Results of Statistical Analysis

Location	**KUET BH(0-5 ft)**	**Khulna BH(0-5 ft)**	**Khulna BH(5-10 ft)**
Standard Deviation	9.86	20.09	18.34
Mean Deviation	8.44	16.71	15.11
Median	53.79	59.5	44.62
Mode	No mode	52	59
C.V. (%)	19.1	34.04	38.94

a) The median value of bearing capacity of soil of KUET at 5 ft depth and Khulna at 5 ft and 10 ft depth is about 53.79 kN/m^2, 59.50 kN/m^2 and 44.62 kN/m^2 respectively.
b) Coefficients of variation of bearing capacity of soil of KUET campus, Khulna BH (0-5 ft), Khulna BH (5-10 ft) are 19.1%, 34.04%, 38.94% respectively.
c) The above results indicate that the variation between the maximum and minimum values of bearing capacity of KUET campus is lower in comparison to Khulna city areas.
d) The results also show that the variation between the maximum and minimum values of bearing capacity at 0-5 ft was lower than that of 5-10 ft.
e) The histogram and other statistical parameters represent the variation of bearing capacity of different locations of Khulna city that help the foundation designer to design foundation safely and economically.

4. CONCLUSIONS

a) Based on the laboratory investigations, information and computations, the estimated average bearing capacity of soil KUET campus at 5 ft depth is 51.60 kN/m^2 and the estimated average bearing capacity of Khulna sub-soil at 5 ft and 10 ft depth are 55.31 kN/m^2 and 47.097 kN/m^2 respectively.
b) The values of bearing capacity of different locations is very low which indicates that shallow foundation is suitable for two or three storied buildings. For multistoried buildings, deep foundation such as pile foundation should be adopted or the soil beneath the foundation should be improved by some suitable method.

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The authors would like to express their thanks to CRTS (Consultancy, research and testing services) and laboratory technicians of Department of Civil Engineering, KUET, for their co-operation and tolerance during this research work. In addition, they wish to express their gratitude to the anonymous reviewers whose suggestions and remarks have greatly helped them to improve the quality of the manuscript.

REFFRENCES

Ansary, M. A., Siddique, A. and Bashar, M. A. (2010). "Geotechnical Characterization of Soil of Dhaka City". International Conference on Civil Engineering.

Das, B.M. (2002). Principles of Geotechnical Engineering. (Fifth edition), Thomson Book House, Canada.

Gupta, S.P. (1983). Advanced Practical Statistics. S Chand & Company Limited, New Delhi.

RAFIZUL ET AL (2006) "Compressibility Properties of Reconsituted organic soils at Khulna Region of Bangladesh." Proceedings of 4th ICSSE, 367-372, Canada.

Spigel, M. R., Schiller, J. and Srinivasan, A. R. (2001). Probability and Statistics. McGraw-Hill, New York.

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INVESTIGATION ON STRENGTH DEVELOPMENT OF CEMENT STABILIZED ORGANIC SOIL

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ABSTRACT

Soil stabilization by mixing chemical admixtures is a recent trend in civil engineering research. The strength developments of natural and stabilized organic soil were investigated in this study. The general practice is to use a shallow foundation for building and other such structure if the sub-soil close to the ground surface possesses sufficient bearing power; however the subsoil in the Khulna region consists of very soft soil up to a considerable depth and the bearing capacity is very low. To investigate the strength and deformation characteristics of stabilized soil and to conduct the experiment of soil stabilization, undisturbed soil was collected from the selected site of the KUET campus from a depth of 12-15ft below the ground surface. The soil sample was brought to the laboratory and tested for Atterberg limit, grain size distribution, compaction test and mixed with cement at pre-specified percentage (2.5%, 5%, 7.5% and 10%). The prepared sample was kept under the water for curing (7days, 14days, 21days and 28days). Afterward unconfined compression test and California Bearing Ratio (CBR) was determined. The unconfined compression test and CBR test was performed at various curing period. Curing period for unconfined compression test was; at initial setting time (8hrs), 7days, 14days, 21days and 28days. Curing period for CBR test was 3days, 7days and 28days. Improvement of unconfined compression strength and CBR values was observed with the increment of the percentage of cement content. It was observed that shear strength (qu) was increased from 103.68 to 546.76kPa and the CBR value was increased from 0.74 to 16%. It was also found that the strength and CBR values improve with the increasing of curing period. The suitability of the stabilization technique was also studied with respect to the cost and strength.

Keywords: *Strength Development, Soil Stabilization, Bearing Capacity, Unconfined Compression Test, California Bearing Ratio (CBR)*

1. INTRODUCTION

Rapid urbanization followed by population growth requires varies types of civil engineering infrastructure and facility service. Because of scarcity of suitable lands it has become difficult to select and appropriate location for infrastructure. At present, for various types of construction engineer are forced to lay foundation on soft ground. The negative effect of soft ground like low strength, stability, high settlement, differential settlement or time-settlement sometimes no choice other than rejection-replacement, excavation and dumping as a waste. To improve the bearing capacity of foundation, subgrade etc. soil stabilization by mixing chemical admixtures (e.g. cement, lime, fly-ash etc.) was studied by many researchers (e.g. Bjerrum and Edie, 1956; Herrin and Mitchell, 1961; Chowdhury R.N. 1972; T.S. Naaraj, N.Miura (Saga University 1977)). They observed in their research that the supporting power of the soil could be significantly increased by stabilizing the loose or soft soil. The subsoil condition in Khulna region consist of recent alluvial deposits and organic composition and it often creates problems to compressibility characteristics of soil. In this study the suitability of organic deposit by mixing cement of different percentage as load bearing soil is investigated.

Stabilized soil can be used as a cut off wall for physical encapsulation to reduce permeability and solidification of the contaminated soil can be used to chemically immobilize contaminants to reduce the potential for pollutant transfer into the environment.

The general practice is to use a shallow foundation for building and other such structure if the sub-soil close to the ground surface possesses sufficient bearing power; however the subsoil in the Khulna region consists of very soft soil up to a considerable depth and the bearing capacity is very low. Therefore, to improve the sub-soil condition, admixture (e.g. cement) if different proportions (e.g. 2.5%, 5%, 7.5%, 10%) were mixed with the natural organic soil and laboratory tests were performed to investigate the strength and deformation characteristics of stabilized soil.

Analysis is done to find out the compressive strength at the optimum range of admixture (e.g. cement) for the improvement of Khulna soil.

2. METHODOLOGY

Portland cement is composed of calcium-silicates and calcium-aluminates that, when combined with water, hydrate to form the cementing compounds of calcium-silicate-hydrate and calciumaluminate-hydrate, as well as excess calcium hydroxide. Because of the cementitious material, as well as the calcium hydroxide (lime) formed, Portland cement may be successful in stabilizing both granular and fine-grained soils, as well as aggregates and miscellaneous materials. A pozzolanic reaction between the calcium hydroxide released during hydration and soil alumina and soil silica occurs in fine-grained clay soils and is an important aspect of the stabilization of these soils. The permeability of cement- stabilized material is greatly reduced. The result is a moisture-resistant material that is highly durable and resistant to leaching over the long term.

2.1 Factors Affects the Soil-Cement Stabilization

Soil-cement stabilization process is affected by many factors. Following are the most important factors that affect the stabilization process.

- Nature of Soil

The effectiveness of cement stabilization also depends on the soil type. However extensive study showed that it is not only the content of the organic matter but also the nature and type of the organic matter affect the properties of cement treated soil. Some organic matter may delay or inhabit the hydration process where other does not affect the reaction at all. Decomposition of organic compound to organic acid due to biological influence also negative effect the cement stabilization.

- Water Content

The water content of any soil is one of the most influential factors for any binder stabilization. If the water content of any soil increases, more cement is required to bring any significance effect.

- Curing Condition

It is well know that temperature influence the strength characteristics of binder treated soil. Increase in temperature accelerates the binder chemical reaction hence strength. In the controlled of a laboratory, temperature can be controlled easily. Temperature condition in the field is quite different from laboratory and varies depending on the binder and measurement position.

- Mixing, Compaction

A stronger and more durable soil-cement will be produced if the soil cement water mixer is more intimately mixed. Mixing will however, a result is decreased strength if it is continued long after cement hydration has begun. The amount of water is depended on the compaction value and adequate amount of water is needed to accelerate the hydration process.

- Admixture

Certain chemical admixture added to soil-cement with the purpose either reduces the cement consumption or to make a soil suitable for stabilization when it is not responsive to cement alone in its natural state. Lime and calcium chloride are commonly used with clays and soils containing organic matter, sodium carbonate and sodium sulphate has also been tried.

2.2 Effect of Cement on the Properties of Soil

- Index Properties

The calcium ion (Ca^{++}) replaces the monovalent ions which are normally attracted to the surface of the negatively charge clay particle. The crowding of calcium ion, clay surface result in the flocculation of clay particles resulting in the increase of coarser soil particle. The plastic limit of soil is also increase with the cement content while the plasticity index decreases.

- Unconfined Compression Strength

Unconfined compressive strength gain with time for various percentage of cement adding with soil. Adding 10% cement increase the unconfined compressive strength from 2.5t/m^2 to 35t/m^2.

- Compressibility Characteristics

The compressibility characteristics of the clay for 10% to 15% cement are extremely small even up to a maximum stress. The effect of cement treatment on the preconsolidation pressure, compression index and the coefficient of consolidation etc.

2.3 Methods of Stabilization May be Grouped As

- Compaction
- Preloading
- Drainage
- Using a vibratory equipment
- Grouting
- Geo-textile
- Admixture

2.4 Experimental Details

To conduct the experiment of soil stabilization, undisturbed soil was collected from the selected site of the KUET campus from a depth of 12-15ft below the ground surface. The soil sample was brought to the laboratory and tested for Atterberg limit, grain size distribution, compaction test and mixed with cement at pre-specified percentage (2.5%, 5%, 7.5% and 10%). The sample was kept under the water for curing (7days, 14days, 21days and 28days). Afterward unconfined compression test and California Bearing Ration (CBR) was determined.

2.6.1 Collection of Sample

Organic soil was collected from a depth of 12-15ft below the existing ground level to find out strength and CBR values of natural soil and cement stabilized soil.

2.6.2 Determination of Atterberg Limit

Atterberg limit test (e.g. liquid limit and plastic limit) of collected natural organic soil were performed in the laboratory to classify the soil used for the stabilization purpose. The details test procedure was carried out as per ASTM D-2487-90.

Table 2.1 Liquid Limit Determination

Can number	1	2	3	4
Number of blows	35	24	19	15
Weight of wet sample + Can	63.6	64.2	63.5	63.1
Weight of dry sample + Can	38.2	38.1	37.3	36.7
Weight of water	25.4	26.1	26.2	26.4
Weight of can	22.4	22.3	22.3	22.1
Weight of dry soil	15.8	15.8	15.0	14.6
Water content (%)	160.76	165.18	174.66	180.82

Table 2.2 Plastic Limit Determination

Can Number	1	21	3
Weight of wet sample + can	26.5	28.3	26.7
Weight of dry sample + can	24.6	25.6	24.6
Weight of water	1.9	2.7	2.1
Weight of can	22.2	22.4	22.2
Weight of dry soil	2.4	3.2	2.4
Water content (%)	79.2	84.37	87.5

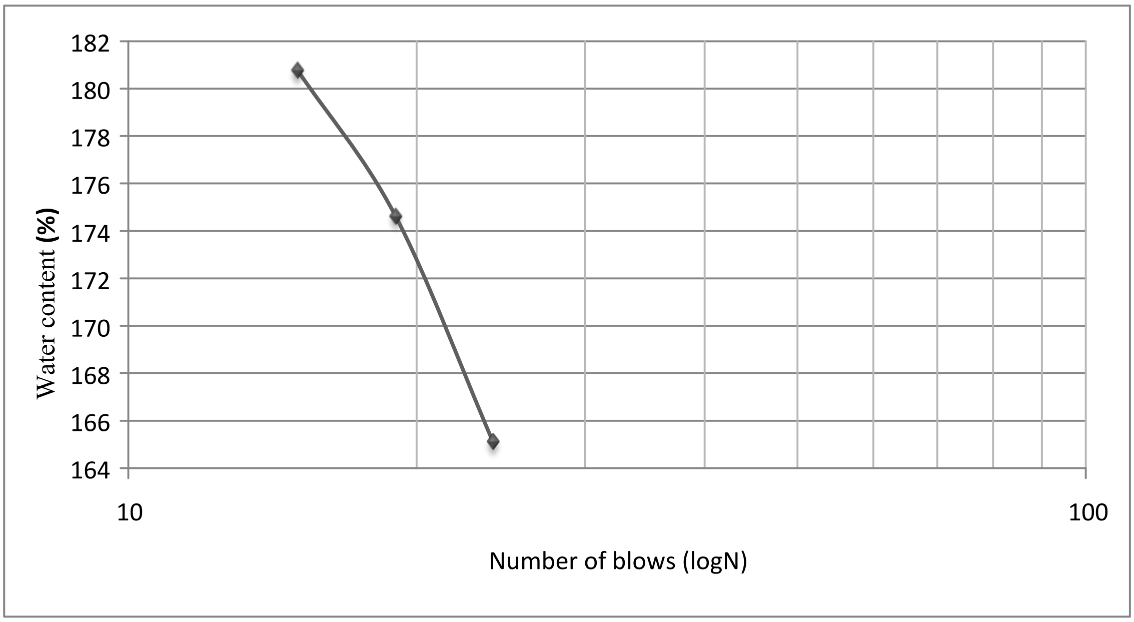

Fig. 2.1 Flow Curve

Table 2.3 Determination of Organic Content

Weight of crusiebole (gm)	35.50
Weight of wet sample + crusiebole (gm)	72.32
Weight of oven dried sample + crusiebole (gm)	58.93
Weight of wet sample (gm)	36.82
Weight of oven dried sample (gm)	23.43
Weight of organic matter (gm)	13.39
Organic content (%)	47

2.6.3 Results (Atterberg Limit)

Liquid limit= 171% Organic Content=47%
Plastic limit= 84%
Plasticity index= Liquid limit – Plastic limit= (171 – 84) %= 87%

2.6.4 Determination of Specific Gravity

Specific gravity of natural soil sample was found as per ASTM D-854-83 Standard

Table 2.4 Specific Gravity of Soil

Weight of pycnometer, W_1 (gm)	153
Weight of pycnometer + dry soil, W_2 (gm)	204
Weight of pycnometer + dry soil + water, W_3 (gm)	677.5
Weight of pycnometer + water, W_4 (gm)	651.6
Temperature (^{0}C)	33
Weight of dry soil, W_S (gm)	51
Weight of equal volume of water, $W_S - (W_3 - W_4)$ (gm)	25.1
$G_S = W_S / (W_S - W_3 + W_4)$	2.03
Specific gravity, G_S	2.03

2.6.5 Grain Size Analysis (Hydrometer Method)

For grain size distribution curve, hydrometer test was performed in the laboratory. It was found that the collected sample was organic silt with high plasticity.

Table 2.5 Grain Size Distribution

Elapsed Time (min)	Temp (^{0}C)	Actual Hydrometer Reading R_a	Correct hydrometer reading R_C	Act/Adj % Finer	Hydrometer correction only for meniscus	L from table 6-5	L/t	K from table 6-5	D (mm)
1	32	40	38.5o	61.6	41	9.6	0.000	0.0142	0.0000
2	32	34	32.50	52.0	35	10.5	5.250	0.0142	0.0320
4	32	28	26.50	42.4	29	11.5	2.875	0.0142	0.0240
8	32	24	22.50	36.0	25	12.2	1.525	0.0142	0.0170
15	32	19	17.50	28.0	20	13.0	0.867	0.0142	0.0130
30	32	17	15.50	24.8	18	13.3	0.443	0.0142	0.0094
60	31	15	12.75	20.4	16	13.7	0.228	0.0144	0.0069
120	31	13	10.75	17.2	14	14.0	0.117	0.0144	0.0049
180	31	12	9.75	15.6	13	14.2	0.079	0.0144	0.0041
300	31	10	7.75	12.4	11	14.5	0.048	0.0144	0.0032
480	32	8	6.50	10.4	9	14.8	0.031	0.0142	0.0025
1440	32	5	3.50	5.6	6	15.3	0.011	0.0142	0.0015

2.6.6 Determination of Optimum Moisture Content

In the laboratory, modified proctor test was performed on remolded sample to determine optimum moisture content and maximum dry density of the natural soil sample. The test was executed according to ASTM D-698 standard.

Table 2.6 Determination of Water Content (Compaction Test)

Sample Number	1	2	3	4	5	6
Wt. of can+wet sample	77.40	78.20	76.80	72.50	78.2	71.7
Wt. of can + dry sample	70.40	69.70	67.30	62.40	64.7	58.2
Wt. of water	7.00	8.5	9.50	10.10	13.50	13.5
Wt. of can	22.50	22.00	22.5	22.2	22.00	22.5
Wt. of dry soil	47.90	47.7	44.8	40.2	42.7	35.7
Water content, w%	14.61	17.82	21.21	25.12	31.61	37.82

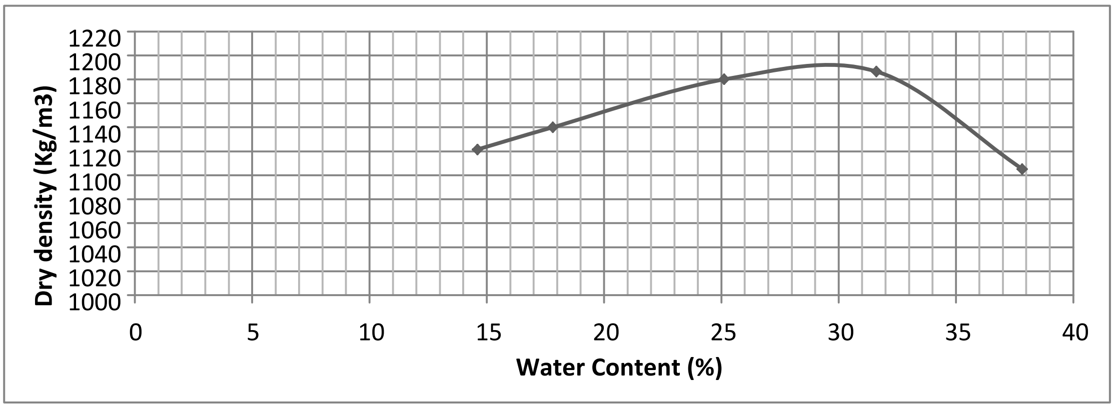

Fig. 2. 2 Determination of Optimum Moisture Content (Dry Density vs. Water Content)

Table 2.7 Determination of Dry Density

Sample Number	1	2	3	4	5	6
Water content, w%	14.61	17.82	21.21	25.12	31.61	37.82
Mass of mold + wet soil (gm)	2969.00	3024.50	3068.30	3152.40	3234.20	3197.40
Mass of mold (gm)	1735.00	1735.00	1735.00	1735.00	1735.00	1735.00
Mass of wet soil, M (gm)	1234.00	1289.50	1333.30	1417.40	1499.20	1462.40
Volume of mold, V (m3)	0.00096	0.00096	0.00096	0.00096	0.00096	0.00096
Bulk density, p= M/V (Kg/m3)	1285.42	1343.23	1388.85	1476.46	1561.67	1523.33
Dry density, p/(1+w) (Kg/m3)	1121.56	1140.07	1145.82	1180.03	1186.59	1105.31

Optimum Moisture Content: 29.5%
Maximum Dry Density: 1190 Kg/m^3

3. RESULTS AND DISCUSSIONS

3.1 Designation of Samples

Cement of various proportion (2.5%, 5%, 7.5% and 10%) were mixed with the natural dry soil with water content 29.5% as obtained from the compaction test of natural organic soil. The samples of natural organic, 2.5%, 5%, 7.5% and 10% are designated as sample S1, S2, S3, S4 and S5 respectively.

3.1.1 Sample S1

Unconfined compression test was performed on five samples of natural organic soil whose curing period were, at initial setting time (8hrs), 7days, 14days, 21days and 28days. The unconfined compression strength for initial setting time (8hrs), 7days, 14days, 21days and 28days are 103.69kPa, 115.91kPa, 124.97kPa, 129.81kPa and 134.26kPa respectively (Fig. 5.1, 5.2, 5.3, 5.4, 5.5). The California Bearing Ratio (CBR) test of the treated soil sample were performed for 3days, 7days and 28days curing time. The California Bearing Ratio (CBR) for 3days, 7days and 28days are 2.73%, 2.82% and 2.82% respectively.

3.1.2 Sample S2

Unconfined compression test was performed on five samples of cement content 2.5% whose curing period were, at initial setting time (8hrs), 7days, 14days, 21days and 28days. The unconfined compression strength for initial setting time (8hrs), 7days, 14days, 21days and 28days are 136.32kPa, 159.34kPa, 173.49kPa, 217.81kPa and 335.38kPa respectively (Fig. 5.1, 5.2, 5.3, 5.4, 5.5). The California Bearing Ratio (CBR) test of the treated soil sample were performed for 3days, 7days and 28days curing time. The California Bearing Ratio (CBR) for 3days, 7days and 28days are 3.06%, 5.18% and 7.94% respectively.

3.1.3 Sample S3

Unconfined compression test was performed on five samples of cement content 2.5% whose curing period were, at initial setting time (8hrs), 7days, 14days, 21days and 28days. The unconfined compression strength for initial setting time (8hrs), 7days, 14days, 21days and 28days are 143.23kPa, 212.32kPa, 232.81kPa, 338.78kPa and 348.85kPa respectively (Fig. 5.1, 5.2, 5.3, 5.4, 5.5). The California Bearing Ratio (CBR) test of the treated soil sample were performed for 3days, 7days and 28days curing time. The California Bearing Ratio (CBR) for 3days, 7days and 28days are 5.58%, 7.47% and 12.97% respectively.

3.1.4 Sample S4

Unconfined compression test was performed on five samples of cement content 2.5% whose curing period were, at initial setting time (8hrs), 7days, 14days, 21days and 28days. The unconfined compression strength for initial setting time (8hrs), 7days, 14days, 21days and 28days are 183.25kPa, 220.54kPa, 253.62kPa, 367.42kPa and 388.93kPa respectively (Fig. 5.1, 5.2, 5.3, 5.4, 5.5). The California Bearing Ratio (CBR) test of the treated soil sample were performed for 3days, 7days and 28days curing time. The California Bearing Ratio (CBR) for 3days, 7days and 28days are 6.4%, 9.54% and 14.56% respectively.

3.1.5 Sample S5

Unconfined compression test was performed on five samples of cement content 2.5% whose curing period were, at initial setting time (8hrs), 7days, 14days, 21days and 28days. The unconfined compression strength for initial setting time (8hrs), 7days, 14days, 21days and 28days are 197.53kPa, 238.95kPa, 306.92kPa, 396.38kPa and 546.76kPa respectively (Fig. 5.1, 5.2, 5.3, 5.4, 5.5). The California Bearing Ratio (CBR) test of the treated soil sample were performed for 3days, 7days and 28days curing time. The California Bearing Ratio (CBR) for 3days, 7days and 28days are 7.25%, 10.38% and 16% respectively.

3.2 Results

Table 3.1: Unconfined compression strength (kPa) values for selected organic soil

Curing Period (days)	**Percentage of cement content (%)**				
	Natural organic soil (0)	2.5	5	7.5	10
At initial setting time (8hrs)	103.69	136.32	143.23	183.25	197.53
7	115.91	159.34	212.32	220.54	238.95
14	124.97	173.49	232.81	253.62	306.82
21	129.81	217.81	338.78	367.42	396.38
28	134.26	330.38	348.85	338.93	546.76

Table 3.2: California Bearing Ratio (CBR) values for selected organic soil

Curing Period (days)	Percentage of cement content (%)				
	Natural organic soil (0)	2.5	5	7.5	10
3	2.73	3.06	5.58	6.4	7.25
7	2.82	5.18	7.47	9.54	10.38
28	2.82	6.94	12.97	14.56	16

Table 3.3: Determination of moisture content during compaction

Sample Number	Mold No. 1 Blows/ Layer: 10	Mold No. 2 Blows/ Layer: 30	Mold No. 3 Blows/ Layer: 65
Can No.	1	2	3
Mass of can + wet soil (gm)	53.36	54.32	54.53
Mass of can + dry soil (gm)	49.43	49.65	49.67
Mass of can (gm)	23.50	22.9	23.5
Mass of moisture (gm)	3.93	4.67	4.86
Mass of dry soil (gm)	25.93	26.75	26.17
Moisture content (%)	15.16	17.46	18.57

Table 3.4: Determination of dry density as molded

Sample Number	Mold No. 1 Blows/ Layer: 10	Mold No. 2 Blows/ Layer: 30	Mold No. 3 Blows/ Layer: 65
Moisture content, w (%)	15.16	17.46	18.57
Mass of mold + wet soil (gm)	7210	7425	7283
Mass of mold (gm)	4240	4243	4240
Mass of wet soil, M (gm)	2970	3182	3043
Volume of mold, V (m^3)	0.00325	0.00325	0.00325
Bulk density, p= M/V (Kg/m^3)	913.84	978.46	936.31
Dry density, p/(1+w) (Kg/m^3)	778	850	789.66

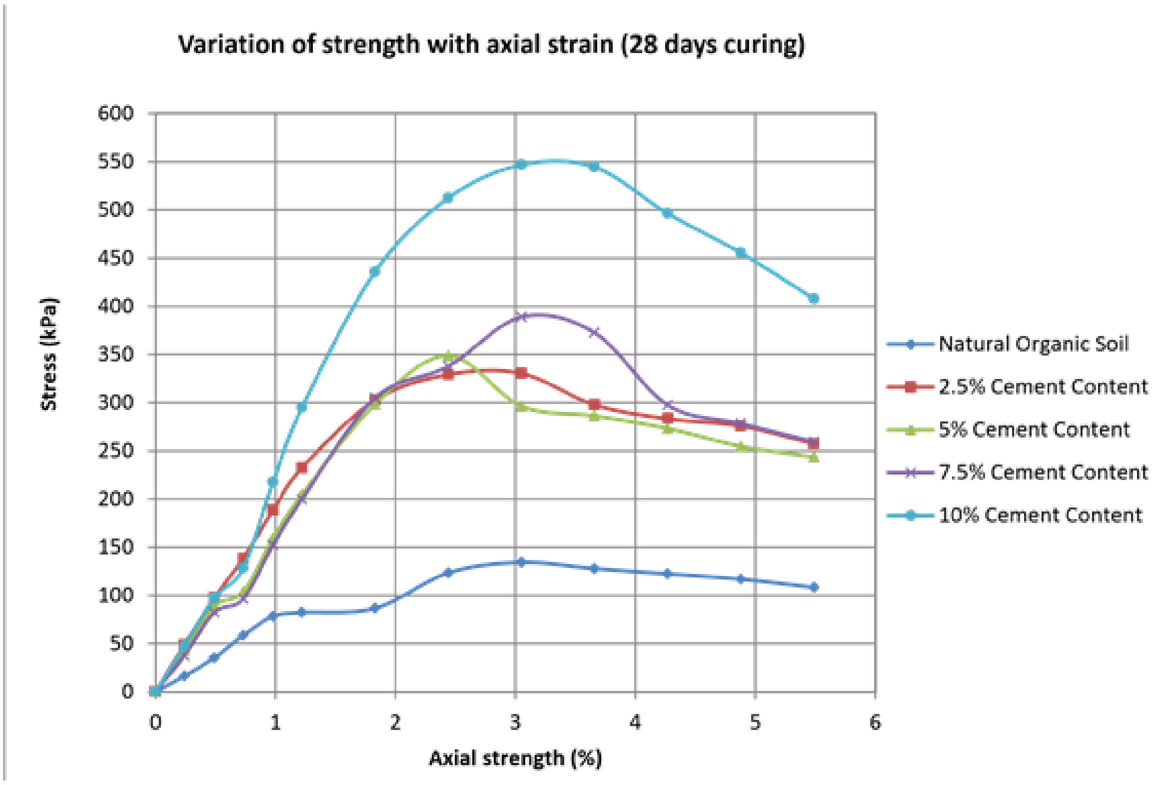

Fig. 3.1 Stress-Strain curve for natural and cement stabilized organic soil at 28 days curing period.

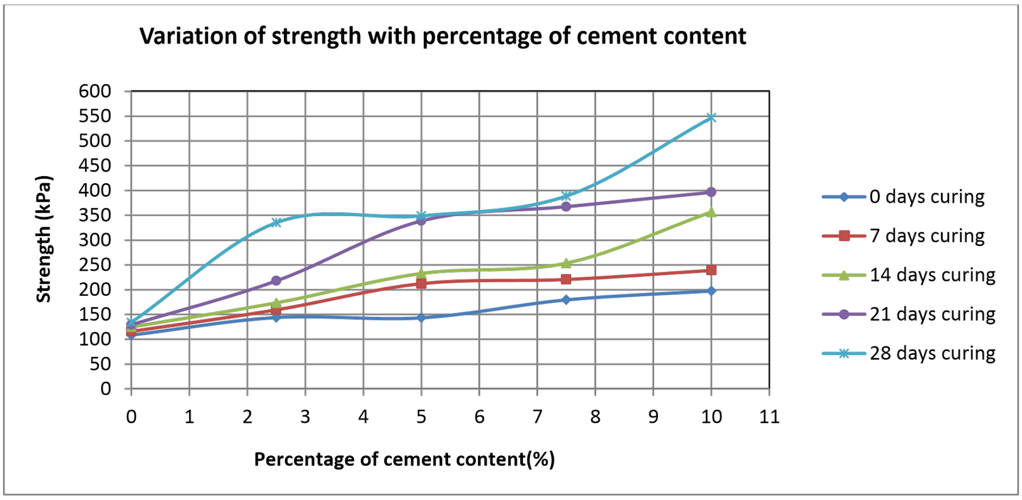

Fig. 3.2 Strength vs. percentage of cement content curve for natural and cement stabilized organic soil at various curing periods.

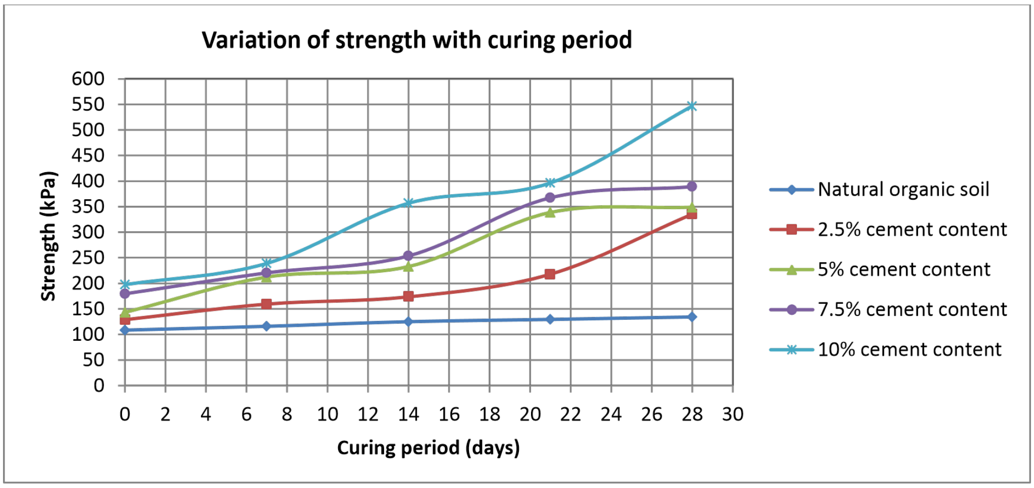

Fig. 3.3 Strength vs. curing periods curve for natural and cement stabilized organic soil at various percentage of cement content.

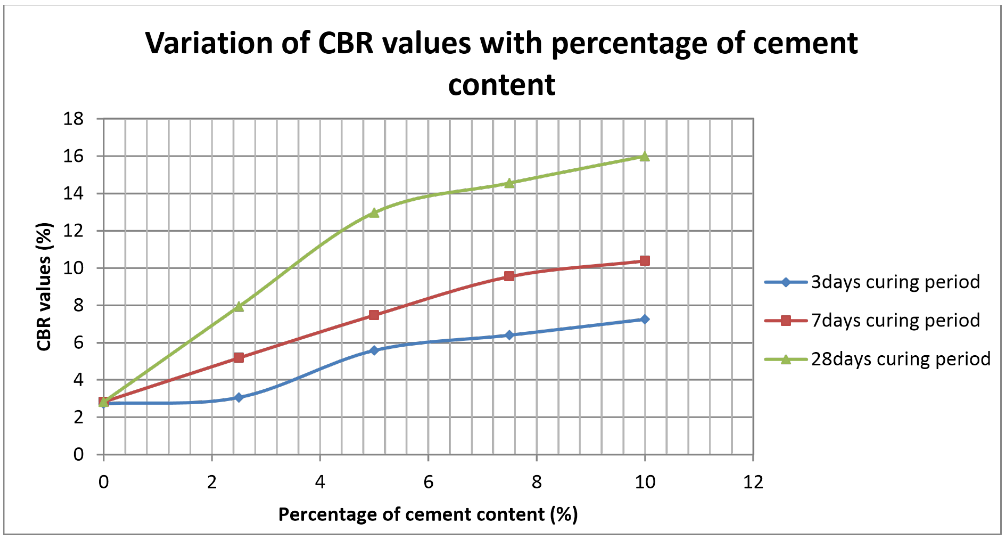

Fig. 3.4 CBR values vs. percentage of cement content curve at various curing period for natural

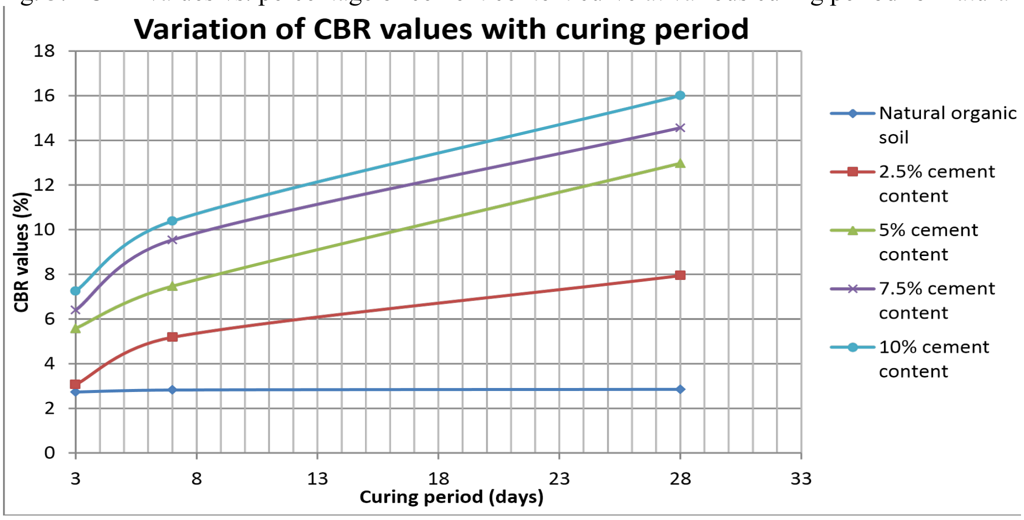

Fig. 3.5 CBR values vs. curing period curve at various cement content for natural and stabilized organic soil.

4. CONCLUSIONS

The unconfined compression strength of stabilized soil as well as the CBR value were increased with the addition of cement content. The increment of compression strength at 28days for 2.5%, 5%, 7.5% and 10% cement content were 140%, 160%, 190% and 307% respectively with respect to natural organic soil. From economical point of view, it was found that 5% cement content is suitable to add for the development of strength by cement stabilization of studied organic soil. Maximum strength and CBR value is developed at 10% cement content. Thus for higher strength 10% cement content is suitable. This study was carried out based on only one location. In future, soil from various locations can be studied to compare and compute the effect of organic content on the strength development of organic soil.

REFERENCES

Bowles, J.E. (1982). Foundation Design and Analysis. McGraw-Hill, New York.

Krebs, R.D. and Walker, R.D. (1971). Highway Materials, McGraw-Hill Book Co, New York.

Sing A. (1992). Geotechnical Engineering, Third edition. CBS publisher and Distributor Pvt. New Delhi -11002.

Ali, M. H., (1993). "Soil Improvement by Preloading – A Case study". First Bangladesh-Japan Joint Geotechnical Seminar on Ground Improvement, Dhaka, Bangladesh, 251-259.

Ali, R. M. E., Karim, M. F and Rahman, M. Z. (2004). "Engineering Geology of Khulna Metropolitan City Area". Bangladesh Journal of Geology, Vol.23, 137-148.

Abboud, M. M. (1973). "Mechanical properties of cement-treated soils in relation to their use in embankment construction", Ph. D dissertation, University of California, Berkeley, CA.

ASTM D4609-01. (2006). Standard guide for evaluating effectiveness of admixtures for soil stabilization, Annual Book of ASTM Standards, 4.08, 757-761.

Currin, D. D., Allen, J. J., and Little, D. N. (1976). "Validation of soil stabilization index system with manual development." Report No. FJSRL-TR-0006, Frank J. Seisler Research Laboratory, United States Air Force Academy, Colorado, 324-336.

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INVESTIGATE THE GEOTECHNICAL PROPERTIES OF EXPANSIVE SOIL USING LOCAL SAND AND RICE HUSK ASH

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ABSTRACT

The geotechnical engineers face difficulties during the construction of geotechnical structures in clayey soils. The engineering properties of those soils may need to be improved to make them suitable for construction. In this research, expansive type of clayey soil from Barind area (Kakon Hat) is considered for investigating the engineering properties. To improve the geotechnical properties of expansive soil the admixture sand and rice husk ash have been used. Sand and rice husk ash are cheap and locally available in Bangladesh. The swelling behavior and Atterberg limits of the untreated and treated (sand and rice husk ash mixed with clay) soil were determined. The results from mixed soil showed that the swelling behavior improved and the liquid limit, plastic limit, shrinkage limit, free swell index and modified free swell index decreased as the percentage of admixture (sand and rice husk ash) increased. On this way the problematic soil may convert into unproblematic soil.

Keywords: e*xpansive soil, admixture, swelling behavior, untreated soil and Atterberg limits.*

1. INTRODUCTION

A difficult problem in civil engineering work exists when the sub-grade is found to be clay. Soils having high clay content have a tendency to swell when their moisture content is allowed to increase. This moisture may come from rains, flood, leaking sewer lines or from the reduction of surface evaporation when an area is covered by a building or pavement. Frequently, these clayey soils cause the cracking and breaking up of pavements, railways, highway embankments, roadways, foundations and channel or reservoir linings. The soils generally have high swell potential and are not favorable when used for construction. Construction damages are possible. When geotechnical engineers are faced with possible construction damage, a need for improving the engineering properties of the soil is justified. Fly ash or pozzolanic materials which are regarded as wastes may be used to make these improvements.

The amount of swelling and shrinkage depends to a larger extent the thickness of the active zone and the degree of saturation. Soft and natural expansive clays are found throughout Bangladesh, U.S.A, India, South Africa, Australia and several other countries in the world. Examples of expansive soils are clay minerals shale, and over consolidated clays. In generals, these soils have potential to expand their original volume by 30% to more than 100%. In shrinkage, soil volume changes could be as high as 65% of original volume. Now a days it has been given higher importance to solve the problems associated with the expansive soils. Techniques are developed in both the field of “Structural Design” as well as “Soil improvement” and researches still going on. Rice husk ash is mainly waste material. Recent research, based on pozzolanic activity, found that rice husk ash was a potential material to be utilized for soil stabilization. Rice husk ash stabilization is the most economical that all other type of soil stabilization technique. In this research work “Ash stabilization” was done using “Rice husk ash” as a stabilizer. Also locally available sand (Padma river sand) used as admixtures in this research.

2. RESEARCH SIGNIFICANCE

The main objectives of the research work are as follows

a) To investigate the engineering properties of expansive soil.
b) To improve strength of expansive soil using different admixture like sand and rich husk ash.

3. THE STUDY AREA

The Barind area is a vast trac. It is situated in the North West part of Bangladesh under Rajshahi district. There are so many villages situated under this trac. The soil characteristics and behavior are different in different villages and different seasons. Kakon Hat, the village is under Barind trac. Expansive soil is available here. So the study area is selected as Kakon Hat. The map of the study area is shown in figure 1.

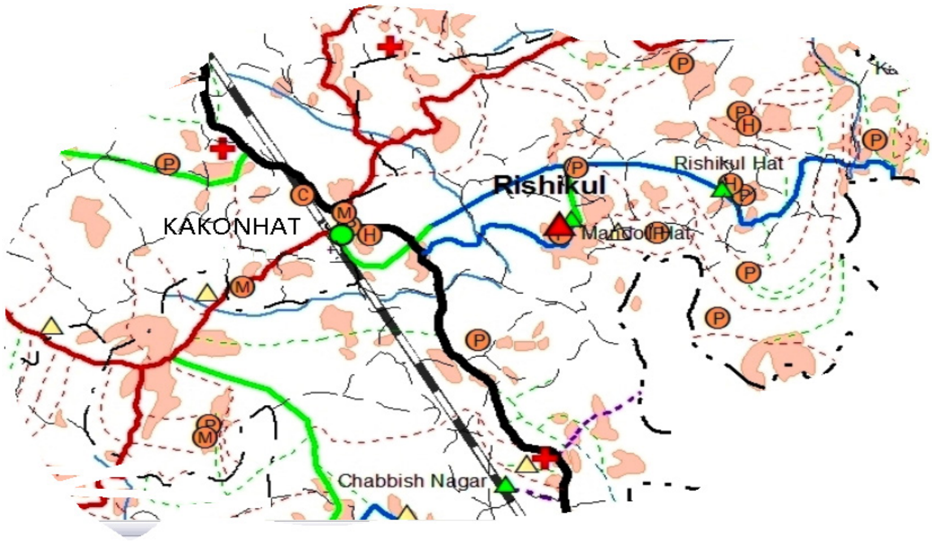

Figure 1: Map of Kakon Hat

4. SOIL STABILIZATION TECHNIQUES

The process of improving engineering properties of soil, particularly the fine grained soil by using admixtures is called soil stabilization. It is required when the soil available for construction is not suitable for the intended purpose. Soil stabilization is used to reduce permeability and compressibility of the soil mass in earth structure and to increase its shear strength.

4.1 Lime Stabilization

The method of soil improvement in which lime is added to the soil to improve its properties is called lime stabilization. The type of lime commonly used in field is hydrated high calcium lime, monohydrated dolomitic lime, calcitic quick lime, dolomitic lime. The quantity of lime used for stabilization of most soils usually is in the range of 5% to 10%. When lime is added in clay soils, several chemical reactions occur, cation exchange and flocculation agglomeration and the monovalent cations generally associated with clays are replaced by the divalent calcium ions.

4.2 Cement Stabilization

The method of soil improvement in which cement is added with in definite proportion in order to improve its properties is called cement stabilization. For clay soils, cement stabilization is effective when the liquid limit is less than about 25. Like lime, cement helps increase the strength of soils and strength increases with curing time. Granular soils and clayey soils with low plasticity obviously are most suitable for cement stabilization.

4.3 Fly Ash Stabilization

The method soil stabilization in which fly ash is mixed with soil as a binding material is called fly ash stabilization. Fly ash is a byproduct of pulverized coal combustion process, usually associated with electric power generation plant. It is a fine grained dust compost of silica, alumina, and various oxides and alkalis. It is a pozzolanic in nature and can react with hydraulic lime to produce cementitious products. For this reason, fly ash lime mixtures can be used for stabilization of highway bases and sub bases. Effective mixes can prepare with 10%-35% fly ash and 2%-10% lime.

4.4 Rice Husk Ash Stabilization

Rice husk ash is mainly waste materials. Recent research, based on pozzolanic activity, found that rice husk ash was a potential material to be utilized for soil stabilization. Rice husk ash stabilization is the most economical than all others type of soil stabilization technique.

4.5 Bituminous Stabilization

Bituminous are non aqueos systems of hydrocarbons that are soluble in carbon di-sulphide. Bituminous stabilization is generally done with asphalt as binder. Any inorganic soil which can be mixed with asphalt is suitable for bituminous stabilization. In cohesionles soils, asphalt binds the soil particles together and thus serves as a bonding or cementing agent. In cohesive soils, asphalt protects the soil bmaintain low moisture content and to increase the bearing capacity.

4.6 Chemical Stabilization

In chemical stabilization, soils are stabilized by different chemicals. The main advantage of chemical stabilization is that setting time and curing time can be controlled. Chemical stabilization is however generally more expensive than other types of stabilization.

4.7 Thermal Stabilization

Thermal stabilization is done either by heating the soil or by cooling it. As the soil is heated, its water content decreased. Electric repulsion between clay particles is decreases and the strength of the soil is increases. Cooling causes small loses of strength of clayey soils due to an increase in inter particles repulsion.

4.8 Electrical Stabilization

Electrical stabilization of clayey soil is done by a process known as electro-osmosis. As a direct current (D.C) is passed through a clayey soil, pore water migrates to the negative electrode (cathode). It occurs because of the attraction of positive ions (cations) that are present in water towards cathode .The strength of the soil is considerably increased due to removal of water.

4.9 Stabilization by Geotextile and Fabrics

The soil can be stabilized by introducing geotextles fabrics which are made of synthetic materials, such as polyethylene, polyester, and nylon. The geotextile sheets are manufactured in different thickness ranging from 10 to v300 mils (1 mil=0.0254). The width of the sheet can be up to 10m.Geotextiles is manufactured in different patterns, such as woven, non-woven, grid and hybrid.

5. TEST PROGRAMS AND PROCEDURES

5.1 Grain Size Distribution

Grain size distribution of a soil sample can be obtained by conducting sieve analysis and hydrometer analysis. Generally sieve analysis is used for soil grained size more than 0.075 mm on the other hand hydrometer analysis is used for soil grained size less than 0.075 mm. For soil samples having both coarse and fine particles both analyses is required for getting the complete particle size distribution. For getting the particle size distribution both hydrometer and sieve analysis were conducted in the laboratory. At first 500 gm of the soil samples were kept submerged under water for 24 hour. After 24 hour the soil samples were sieved through 200 no. sieves (0.75mm) using water jet. The soil retaining on 200 no. sieves after drying was used for sieves analysis. Soil, which passes the 200 no. sieves was collected in a large container and was allowed to settle. These settled soils after drying was used in hydrometer analysis.

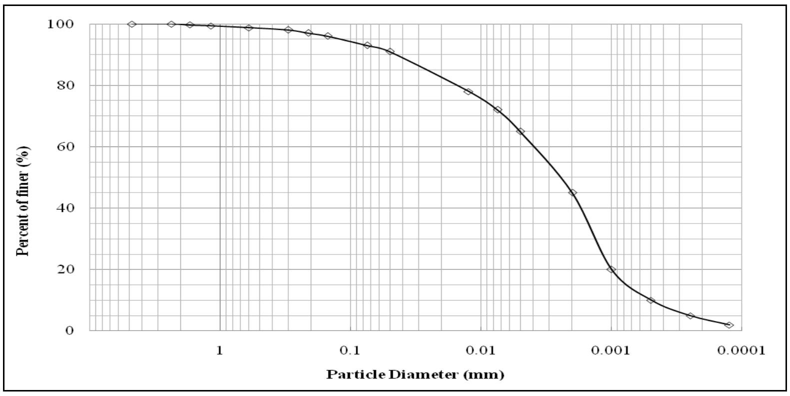

Figure 2: Grain size distribution of soil sample

5.2 Liquid Limit Test

The liquid limit test were conducted for two soil samples refered Sand Treated Soil (STS) and Rice Husk Ash Treated Soil (RHATS). The basic properties of sample specimen's were presented in Table 1. To identify the liquid limit, liquid limit test is carried out for both sample with 0%, 2.5%, 5%, 7.5% and 10% admixtures. About 120 gm for the specimen was taken, passing through the 425-micron sieve and it was mixed thoroughly with distilled water in the evaporation dish or on the marble plate so that uniform paste was formed. The soil was left for sufficient time so that water may permeate through out of the soil mass in the case of flat clays , this maturing time may be up to 24 hours. For an average soil through mixing for about 15 to 30 minutes may be sufficient. The amount of water to be added depends on the type of soil and is a matter of experience. A portion of the paste was taken with the spatula and was placed in the center of the cup so that it is almost half filled. Then the cup of the wet soil was leveled symmetrically with the spatula, so that it was parallel to the rubber base and the maximum depth of the soil was 1 cm. With the help of Grooving tool, the paste in the cup was divided along the diameter (through the centre line of the cam follower), by holding the tool normal to the surface of the cup and drawing it firmly across. Thus, a V-shaped gap of 2 mm wide at the bottom and 13.6 mm at the top and 8 mm in depth was formed. The handle of the apparatus was turned at the rate of 2 revolutions per second, until the two parts of the soil came in contact with the bottom of the grove along a distance of 10 mm. The number of blows required to cause the groove close for approximate length of 10 mm was recorded. A representative slice of soil was collected by the widthwise from one to the soil cake at right angles to the groove , including the portion of the groove of the soil flowed together and put in an air tight container, its water content was determined. The remaining soil from the cup was removed and mixed with the soil left earlier on the marble plate (or evaporation dish). Either adding more water or leaving the soil paste to dry, as the case may be changed the consistency of the mix. The number of revolution to close the groove was noted and the soil was kept water content determination.

5.3 Plastic Limit Test

The plastic limit test are conducted for two soil samples refered STS and RHATS. The basic properties of sample specimen's were presented in Table 1. To identify the plastic limit, plastic limit test is carried out for both sample with 0%, 2.5%, 5%, 7.5% and 10% admixtures. To carry out this test a sample of soil is about 20 gm of air dried of the material passing 425 sufficient distilled water to plastic soil mass was left for mass may be left to stand for mass. Soil sample was taken from the thoroughly mixed portion 425 micron sieve . It was mixed on the marble plate with make it plastic enough to be shaped in to a ball. Give some time to mature. In some fat clays. The plastic soil 24 hours to allow water to penetrate throughout the soil . About 8 gm of the plastic soil was taken a ball was made of it and it was rolled on the marble (or glass) plate with the hand with just sufficient pressure to roll the mass in to a tread of uniform diameter throughout its length. When the diameter of the thread was decreased to 3 mm the specimen was kneaded together and rolled out again. The process was continued until tread just crumbled at 3 mm diameter. The crumbled soil tread was collected in the air tight container and was kept it for water content determination. The test was repeated twice more. Thus three readings were obtained for the determination.

5.4 Shrinkage Limit Test

The shrinkage limit test were conducted for two soil samples refered STS and RHATS. The basic properties of sample specimen's were presented in Table 1. To identify the shrinkage limit, shrinkage limit test is carried out for both sample with 0%, 2.5%, 5%, 7.5% and 10% admixtures. To carry out this test, a sample of soil is about 100 gm of soil sample was taken from a thoroughly mixed portion of the material passing 425 micron IS sieve. About 30 gm of the above sample was placed in evaporating dish and it was mixed thoroughly with distilled water. Sufficient water was added to fill the void in the soil completely and to make the soil pasty enough to be readily worked into the shrinkage dish without entrapping air bubble . In the case of plastic soil the water content of the paste may exceed its liquid limit by as much as 10%. While for friable soil the amount of water required to obtain the desired consistency may be equal to or slightly greater than the liquid limit. Determination of mass and volume of the shrinkage dish. A shrinkage dish was cleaned its mass determined accurate to 0.1 gm to determine its volume, the shrinkage dish was placed in the evaporating dish and it was filled by overflowing the mercury. The excess mercury was removed by pressing the plane glass plate firmly on its top, taking care that no air entrapped. Any mercury, which was adhering to the outside of the shrinkage dish, was wiped out carefully transferred to another evaporating dish and then the mass of mercury was determined accurate to 0.1 gm. The mass of the mercury divided by its density gave the volume of the shrinkage dish which was also the volume of wet soil pat. Filling the shrinkage dish soil pot. The inside of the shrinkage dish was coated with thin layer of silicon grease or Vaseline. In the centre of the dish, the soil paste was placed about one-third of the volume of the dish, with the help of spatula. The dish tapped gently on a firm surface, cushioned with layers of blotting paper or rubber sheet to allow the paste to flow towards the edges. Another equal installment of the paste in the dish and it was made to flow towards the edges by trapping. Tapping was continued till the paste was compacted and all the entrapped air was brought to the surface .The process was repeated till the dish was completely tilled and the excess soil overflowed . The excess soil paste was stroke off with a straight edge. The soil adhering to the out of the dish was wiped off. The shrinkage dish was kept open to air until the color of pat turned from dark to light. The shrinkage dish was kept in the oven and thus the pat was dried to constant mass at 105^0 to 110^0 C the dish was cooled in a desiccator's and wetted immediately. To determine the volume of the dry soil pat the glass cup kept in the evaporating dish. The cup was filled to overflowing with mercury. The excess mercury was removed by pressing the glass plate with the three prongs firmly over the top of the cup. The cup was transferred carefully to another evaporating dish, if any mercury was adhering to the outside of the cup was wiped off carefully. The oven dried soil pat was placed on the surface of mercury in the cup and the pat was forced carefully into the mercury by pressing it by same glass plate containing three prongs. The plate was pressed firmly on the top of the cup. The displaced mercury was collected carefully and its mass taken to an accuracy of .01 gm the volume of the soil pat then determined by dividing this mass by the density of mercury.

5.5 Unrestrained Swell Test

After cleaning the specimen ring the soil specimen placed in the ring. Assemble the odometer with the soil specimen and porous stone at top and bottom of the specimen. Mount the mould assembly on the loading frame and center is such that load applied in axial. Position the dial gauge to measure vertical swell of the specimen. Apply a small surcharge load 1 lb/in^2 (9.6 kN/m^2). Connect the mould assembly to the water reservoir and the sample is allowed to saturate. The expansion of the volume of the specimen is measured until equilibrium is reached .The percentage of free swell is than calculated. For the treated soil different amount of admixture is added with the dry dust soil (passing through 425 micro sieve). Water (field water content) is added with it and to make a paste. Then the paste is keep 24 hours without any disturb without any loss of moisture.

5.6 Modified Free Swell Index

Taking about 50 gm of soil and dry in a oven at 110^0 for 24 hours. Then make the sample as soil dust using wooden hammer and passes through 425 micro sieve. Taking 10 gm of dust sample and transferred into a 100 ml graduated measuring cylinder containing distilled water. After 24 hours, the swollen sediment volume is measured . Then the modified free swell index is calculated.

6. RESULTS AND DISCUSSIONS

The basic properties of untreated and treated soil obtained from grain size distribution curve, pycnometer test and modified proctor test is presented in table 1.

Table 1: Basic properties of soil sample

Properties			Obtained Value
Specific gravity of untreated:			2.65
Grain size analysis			
D_{10} (mm)			0.005
D_{30} (mm)			0.015
D_{60} (mm)			0.004
Coefficient of uniformity, Cu			2.66
Coefficient of curvature, Cc			1.125
Fineness Modulus, F.M			1.23
Compaction test result for untreated soil:			
Maximum dry density, $\Upsilon_{d(max.)}$ (gm/cm^3)			1.81
Optimum moisture content, (%)			12.50
Void ratio, e			0.50
Specific gravity of treated soil:			
0% sand mixed with soil	2.65	0% rice husk ash mixed with soil	2.65
2.5% sand mixed with soil	2.66	2.5% rice husk ash mixed with soil	2.62
5% sand mixed with soil	2.67	5% rice husk ash mixed with soil	2.58
7.5% sand mixed with soil	2.68	7.5% rice husk ash mixed with soil	2.56
10% sand mixed with soil	2.70	10% rice husk ash mixed with soil	2.53

As the specific gravity of sand is greater than the clay, the specific gravity of sand treated soil increased with the increasing of percentage of sand is presented in table 1. The specific gravity of rice husk ash is less than the clay. That's why the specific gravity of rice husk ash treated soil decreased with the increased of percentage of rice husk ash. Sand and rice husk ash are locally available materials and cheap to other stabilization techniques. The experimental investigations are carried out in the two different admixtures. The tests were performed for varying percentages of admixtures such as 0%, 2.5%, 5%, 7.5% and 10%. From the test result it is clearly seen that, liquid limit (LL), plastic limit (PL) and shrinkage limits (SL) decreased as the percentage value of sand and rice husk ash increased. Modified free swell index (MFSI) and unrestrained swell index (USI) decreased with increased the percentage value of sand and rice husk ash (RHATS). All the specimens were tested as per standard specification with and without admixture of various proportions. The test results values are presented in table 2 to 5. The liquid limit, plastic limit and shrinkage limit of STS and RHATS decrease with the increases of admixtures shown in table 2, table 3 and table 4. The unrestrained swell and free swell index of STS and RHATS also decreased with the increases of admixtures shown in Table 5 and Table 6.

Table 2: Behaviour of liquid limit with respect to various percentage of sand and rice husk ash

Percentage of Admixture	Liquid limit of STS (%)	Liquid limit of RHATS (%)
0.0	32.00	32.00
2.5	30.00	28.00
5.0	27.00	26.00
7.5	25.00	23.00
10.0	22.00	19.00

Table 3: Behaviour of plastic limit with respect to various percentage of sand and rice husk ash

Percentage of Admixture	Plastic limit of STS (%)	Plastic limit of RHATS (%)
0.0	21.56	21.56
2.5	21.00	19.00
5.0	19.00	16.00
7.5	18.50	15.50
10.0	17.00	15.00

Table 4: Behavior of Shrinkage limit with respect to various percentage of sand and rice husk ash.

Percentage of Admixture	Shrinkage limit of STS (%)	Shrinkage limit of RHATS (%)
0.0	17.00	17.00
2.5	15.00	16.00
5.0	12.00	14.00
7.5	10.00	13.00
10.0	8.00	11.00

Table 5: Behavior of Unrestrained Swell with respect to various percentage of sand and rice husk ash.

Percentage of Admixture	Unrestrained swell of STS (%)	Unrestrained swell of RHATS (%)
0.0	1.70	1.70
2.5	1.32	1.28
5.0	1.10	1.18
7.5	0.95	1.00
10	0.70	0.68

Table 6: Behaviour of Modified free swell index with respect to various percentage of sand and rice husk ash

Percentage of admixture	Modified free swell index of STS (%)	Modified free swell index of RHATS (%)
0.0	2.86	2.86
2.5	2.75	2.55
5.0	2.40	2.11
7.5	1.98	2.05
10.0	1.74	1.90

7. CONCLUSIONS

The Liquid limit, plastic limit and shrinkage limit decreased as the percentage of admixtures increased. Atterberg limit decreases, thus the geotechnical properties of the soil improved. The swelling behaviour of soil improved for adding varying percentage of admixtures. The settlement of engineering structures depends on the swelling behaviour. Therefore, to reduce the settlement of structures this improvement techniques may be used. On this way the problematic soil converts into un problematic soil.

REFERENCES

Bowles, E.J. (1998). *Foundation Analysis and Design,* 4th edition. McGraw Hill Book Co. Singapore.

Budhu, M. (2000). *Soil Mechanics and Foundations.* John Wley & Sons, Inc. New York.

Bardet, J. P. (1997). Experimental Soil Mechanics, Prentice-Hall, Inc, United States.

Craig, R. F. (1997). *Soil Mechanics*, 6th edition, Spon Press, London

Cernica, J. N. (1995). *Geotechnical Engineering: Soil Mechanics*, John Wiley & Sons, Inc, United States.

Das, M. B. (1983). *Advanced Soil Mechanics*, Hemisphere Publishing Corp., McGraw Hill, London.

Das, M. B. (2005). *Fundamentals of Geotechnical Engineering*, Nelson, Toronto, Canada.

Das, M. B. (1998). *Principle of Geotechnical Engineering*, 4th edition. PWS Publishing corp., United States of America.

Kolbuszewski, J. and Frederick, M.R. (1963). *The significance of particle shape and size on the mechanical behavior of granular materials*, European Conference on Soil Mechanics and Foundation Engineering, Vol. 1.

Kirkpatrick, W. M. (1965). *Effects* of *grain size* and *grading* on the *shearing behavior* of *granular materials, Proceedings of 6th ICSMFE, Vol. 1, pp. 273-277.*

Koerner, R.M. (1970). *Effects of Partical Characteristcs on Partical Strength*, Journal of Soil Mechanics and Foundations Division, ASCE, Vol. 96, SM4, 1221-1234.

Liu, C. & Evett, B.J. (1998). *Soils and Foundations*, 4th edition, Prentice-Hall, Inc, United States of America.

Mehmet S. O. (2008). *Shear Strength Behavior of Sand - Clay Mixtures.* Thesis paper, Master of Science in Civil Engineering Department, Middle East Technical University, Ankara, Turkey.

Yasin, M.J.S. & Saifullah, M.M.A. *Effect of Particle Characteristics On The Strength And Volume Change Behavior Of Sand*, Journal of civil engineering, the institution of engineers, Bangladesh. Vol. CE 31, No.2, 2003.

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SLOPE STABILITY ANALYSIS OF AN EMBANKMENT AT JAMUNA RIVER

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ABSTRACT

A study on Slope Stability Analysis of an embankment has been carried out considering different slopes at different conditions. For this purpose embankment soil has been collected from Basuria in Sirajganj near the bank of Jamuna River. Grain size analysis of the sample reveals that it contains 63% sand, 35% silt and 2% clay. The cohesion and angle of internal friction are found to be 7 kPa and 21° respectively from shear test. For the parametric study, shear strength parameters have been modified to be 10 kPa; 14° and 20 kPa; 12°. Based on the results of soil investigations, stability analysis using STB2010 at some conditions (dry, high flood level, low flood level and rapid drawdown with slope 1:1, 1:1.5 and 1:2) of the embankment has been performed. It has been found that the safety factor decreases with steep slope while increasing with flatter one. The maximum safety factor has been found 2.255 for soil at dry condition with a slope 1:2 while the minimum factor is 0.66 at rapid drawdown condition with 1:1 slope. The soil having minimum factor possesses very bad condition which needs to be protected with a conventional design solution, Revetment Design.

Keywords: *Embankment, shear strength, Slope Stability*

1. INTRODUCTION

Over the last few decades, nearly 13000 km of flood and river embankments have been repaired in Bangladesh (Hossain, M.Z. and Sakai, T., 2011). But, earthen embankments in Bangladesh are facing problems like erosion, breaching in every year. The major causes of failure identified were breach of the embankment cutting by the public, overflow, erosion, seepage and sliding. Furthermore, insufficient supervision during construction results in poor-quality earthworks with the use of inappropriate soil materials, insufficient or no clod breaking, inadequate compaction and or insufficient laying of topsoil layers, the use of inferior materials, inadequate maintenance, river migration and cutting by the public (Hoque and Siddique, 1995). The stability of earthen embankments is influenced by seepage occurred during the increase and decrease of the adjacent water level in the river or reservoir (Morii and Kunio, 1993). In Bangladesh, nearly 4,600 km of embankments along the bank of big rivers are flowing across the country. JAMUNA, one of the big rivers is flowing alongside of Sirajganj district of Bangladesh (Figure 1). At 41 locations of its bank, the length of failure occurring is about 160.62km.

The concept of stability is one of the most important issues in Civil Engineering field. Stability concept comprises some of the important factors in Civil Engineering namely: force, moment and equilibrium. The factors contributing to the slope stability include: the type of soil, geometry of the cross-section of the slope, weight, loads and load distribution, gravity, increase in moisture content of the soil material, decrease in shear strength of soil, vibrations and earthquakes, due to human action like excavation, undercutting and overloading.
Revetment Design is the most conventional and gratifying solution for river bank protection (http://www.kennisbank-waterbouw.nl/DesignCodes/rockmanual/chapter%208.pdf). Revetments are used to protect banks and shorelines from erosion caused by waves and currents. It is assumed to be easily accessible for Bangladesh. Revetment design using concrete block is considered to be economical rather than using other materials. Articulating concrete blocks (ACBs) are designed to provide stability and erosion control in a wide variety of hydraulic applications. Made on dry cast block machines, the individual units are engineered to capitalize on the weight of concrete, friction between units, and the interconnection of units into flexible mattresses. Flexibility between units is provided to allow the mat to conform to minor deformations in the sub grade. Classes of individual units can be produced at varying thicknesses, providing the designer flexibility in selecting appropriate levels of protection. The range of block classes allows selection of the proper combination of unit weight, surface roughness, and open area for hydraulic stability. For example, an Armor Flex armor unit, shown in Figure 2, is substantially rectangular, having a flat bottom to distribute the weight evenly over the sub grade.

As Slope stability is influenced by physical features of the embankment, for different types of embankment and embankment geometry many comprehensive studies have been conducted in the developed countries and reported in their research reports (Flate and Preber, 1974; Mesri et al, 1994; Olson 1998). This paper is aimed to determine the stability and settlement characteristics of Jamuna embankment at selected conditions. It presents a study on the investigation of physical and mechanical properties of Jamuna river embankment materials located at Basuria in Sirajganj district of Bangladesh. Attempt has also been made to evaluate the existing design methodology for embankment stability analysis through a case study.

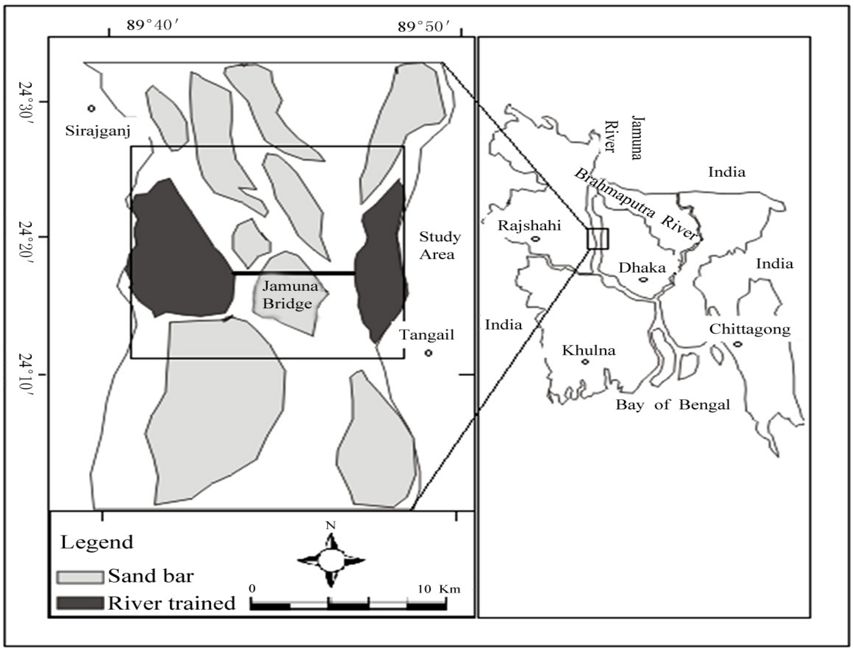

Figure 1: Location map of the study area.

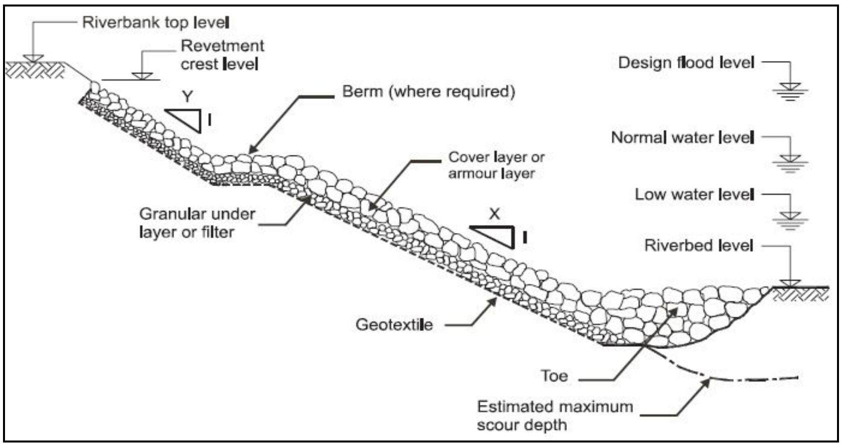

Figure 2: Components of typical armor stone revetment.

2. GEOLOGY OF THE STUDY AREA

The study Jamuna River in 1787 a tectonic movement followed by an abnormal flood led changes in the course of the Brahmaputra and started its flow through a new course known as the Jamuna (Bhuiyan, M.A.H., Rakib, M.A., Takashi, Rahman, M.J.J. and Suzuki, Shigeyuki, 2010). It is the main channel of the Brahmaputra River when it flows out of India into Bangladesh. Jamuna enters in Bangladesh from the North West side of Kurigram district and flows to south, ending its independent existence as it joins the Padma River near Goalundo Ghat. Bounding coordinates of the river area is W: 89.532, E: 89.871, N: 25.228, S: 23.869. The climate of the study area is tropical monsoon. Jamuna Bridge site is located between Tangail and Sirajganj town. It lies within latitude 24022′50″–24026′30″N and longitude 89055′30″–89058′45″E. The river reach is characterized by well defined braiding nature, meta-stable islands, nodes, sandbars, shifting ana-branches and rigorous bank erosion.

Geomorphologically, the eastern bank is bounded with the lateral extension of Madhupur Tract and the west bank is the Barind Tract, which is composed of silty clay. During monsoon, the average annual discharge of Jamuna River (JR) at Ba-hadurabad point is about 50000 m3/s. However, the discharge increased to 100000 m^3/s during the 1988 and 1998 flood events. The average water surface slope is approximately 6.5 cm/km for the lower reach.

The soil deposits mainly consist of the following types of soils: (after Geological Map of Bangladesh, GSB, 1990)

ASL – Alluvial Silt – Light to medium – grey, fine sandy to clayey silt. Commonly poorly stratified; average grain size decreases away from main channels. Chiefly deposited in flood basins and inter stream areas. Unit includes small back swamp deposits and varying amounts of thin, inter stratified sand, deposited during episodic or unusually large floods. Illite is the most abundant clay minerals. Most areas are flooded annually. Included in this unit are thin veneers of sand spread by episodic large floods over flood – plain silts. Historic pottery, artifacts and charcoal (radiocarbon dated 500 – 6000 years B.P.) found in upper 4m.

3. SAMPLE COLLECTION AND LABORATORY TESTS

The soil samples were collected directly from the broken part of the right bank embankment of Jamuna River at Basuria in Sirajganj district. The field investigations consisted of drilling of boreholes, identification of subsoil layer, assessment of density and consistency of subsoil layers by carrying out Standard Penetration Test, collection of disturbed and undisturbed tube samples. One borehole was drilled at Basuria site on the bank of Jamuna River. Geologic profile of the subsoil is made from the bore log data at Basuria site. The soil overall the whole depth possesses non – plastic behavior. Silt with little clay, brown in color, having very loose density exists near the top of the ground surface extending to about 8 feet depth. The SPT – N value of this type of soil at the given depth is 1. Very fine sand with little silt trace mica is encountered just below the top clay layer having grey in color and non-plasticity behavior. It extends up to the final depth of boring 102 feet (30m) and possibly beyond. The density varies with depth such as loose density up to 28 feet depth, medium density up to 63 feet depth and dense density for the rest. At the layer up to 28 feet depth, the average SPT – N value is 3 while the range of SPT – N value is 15 – 25 up to 63 feet depth. Again SPT – N value up to 73 feet depth is 26 and up to 102 feet the range is 28 – 32 feet. Polythene bags have been used for the storage of sample for laboratory tests. The laboratory tests have been conducted in the Laboratory of Bangladesh University of Engineering And Technology. The testing procedures are in accordance with American Society for Testing and Materials (ASTM). The following tests were conducted in the laboratory in order to access the collected sample: Grain size analysis, Compaction test, Consolidated – Undrained Direct Shear test (CU test).

4. LABORATORY TEST RESULTS AND DISCUSSION

Figure 3 and 4 shows grain size distribution of sandy soil and moisture content vs.dry density relationship of sandy soil for Standard Proctor Compaction test respectively. Figure 5 and 6 show Shear stress vs. displacement curve and Shear stress vs. normal stress graph of soil ($c = 7$ kPa, $\varphi = 21°$) for Consolidated undrained direct shear test.

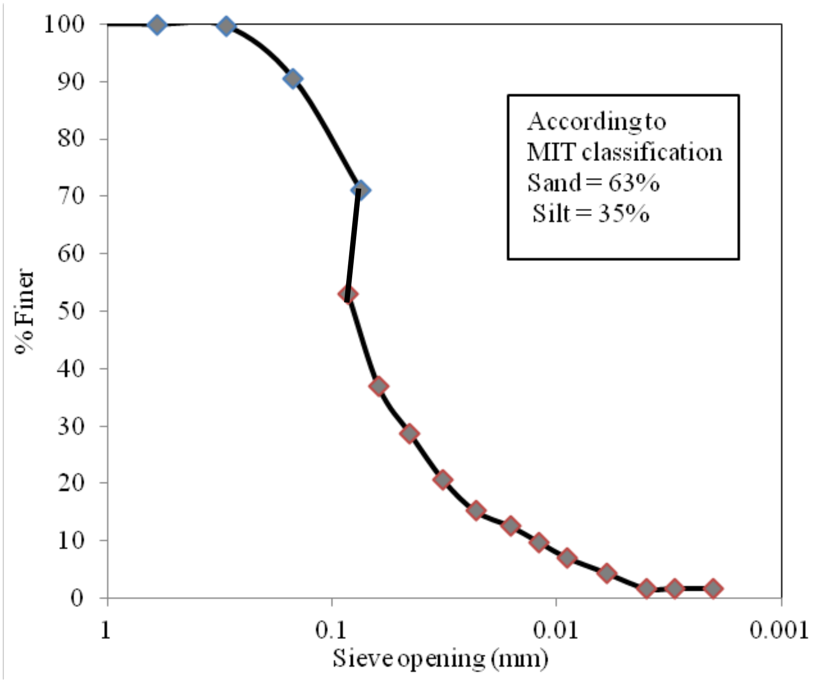

Figure 3: Grain size distribution of sandy soil.

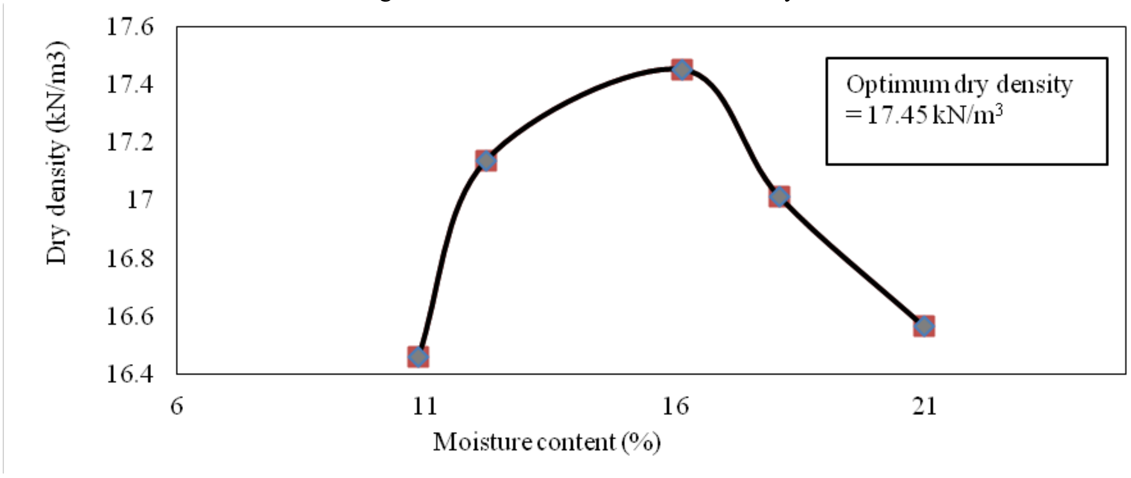

Figure 4: Moisture content vs.dry density relationship of sandy soil for Standard Proctor Compaction test.

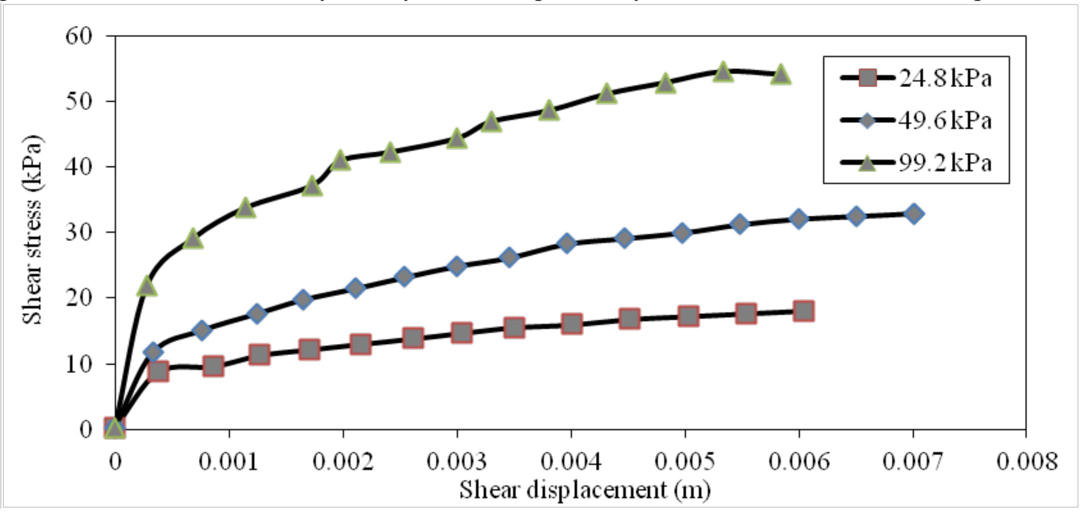

Figure 5: Shear stress vs. displacement curve for Consolidated undrained direct shear test of soil (c = 7 kPa, φ = 21°).

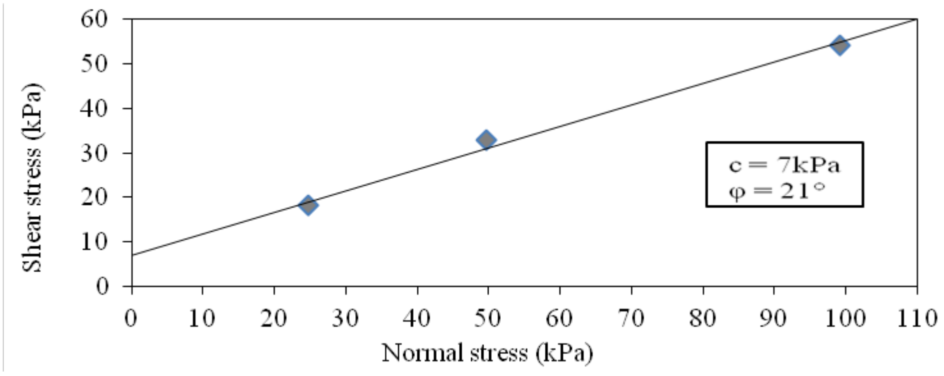

Figure 6: Shear stress vs. normal stress graph of soil (c = 7 kPa, φ = 21°)

According to MIT classification the percentage of sand, silt and clay at our soil sample have been found 63%, 35% and 2% respectively. Hence it can realize that the sample is not the pure sand. According To Unified Soil Classification system (USCS) the soil sample is SW-SM. It has a portion of silt and clay. From the shear test the values of cohesion and angle of internal friction have been obtained for sand. For further parametric study, shear strength parameters have been modified. Table 1 shows the values of cohesion and angle of internal friction of three types of soil obtained from Consolidated Undrained direct Shear Test.

Table 1 Results from Consolidated Undrained direct Shear Test

Soil sample	Cohesion, c (kN/m^2)	Angle of internal friction, φ(degree)
Type 1	7	21
Type 2	10	14
Type 3	20	12

From the table 1 we can analyze that cohesion is increasing with the increase of clay while angle of internal friction is decreasing along with it. The more the cohesion the more would be the presence of clay while the reverse case happens for angle of internal friction. The values of cohesion and angle of internal friction have been used in the stability analysis. The shear strength parameters of the underlying soils have been obtained using the existing correlation with SPT-N value shown in the bore logs.

5. SLOPE STABILITY ANALYSIS

Civil engineers are often expected to make calculations to check the safety of natural slopes, slopes of excavations, and compacted embankments. This check involves determining the shear stress developed along the most likely rupture surface and comparing it with shear strength of the soil. This process is called slope stability analysis. The most likely rupture surface is the critical surface that has the minimum factor of safety.

THE PROGRAM STB2010

This is a program for the analysis of the stability of slope (Verruijt, A., Delft University, 2010). The program uses Bishop's simplified method with some modifications introduced at GeoDelft and the Delft University for the calculation of the facto of safety of circular slip surface, with Koppejan's correction for very deep circles, and a modification to account for the strength reduction of a double sliding model. The program also allows for a possible horizontal body force, to simulate the effect of an earthquake. The soil properties used in the program are:

- W_d: Dry unit weight (kN/m3).
- W_s: Saturated unit weight (kN/m3).
- K_o: coefficient of neutral horizontal stress (-).
- c: Cohesion (kN/m^2).
- phi: angle of internal friction (degrees).
- P/F: switch for the groundwater condition (-).
- p = 0: Zero level of the pore water pressure (m).
- cap: Thickness of capillary zone, above groundwater table (m).
- The unit weight of water is 10 kN/m^3.
-

6. RESULTS OF STABILITY ANALYSIS USING STB2010

STB2010 is used for the analysis of stability of slope, using Bishop's method with some conditions. Figure 7 shows the results of Stability Analysis of soil (c = 7 kPa, φ = 21°) at one of the four conditions (dry condition) with slope 1:1.5.

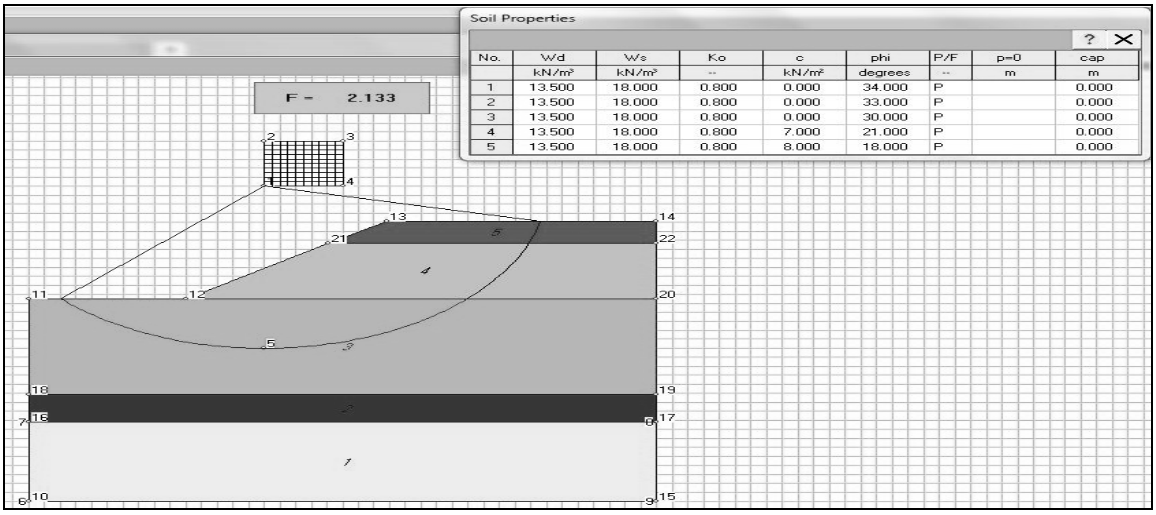

Figure 7: Result of stability analysis of soil (c = 7 kPa, φ = 21°) at dry condition for slope 1:1.5

The results obtained from the analysis using STB2010 are shown in Table 2 to 4. The fluctuation of the safety factor along with four conditions and three slopes for the soils having different parametric characteristics has been shown in Figure 8 to 10. From the figures for all kinds of soil, it has been realized that the safety factor increases with the increase of water due to seepage into the soil. But at rapid drawdown condition the safety factors have been found to be least among all conditions. It causes due to rapid reduction of external water level. Figure 11 and 12 show the variation of safety factors at high flood level and rapid drawdown condition for different soil condition respectively.

Table 2 Results of Stability Analysis of Soil Sample Type 1 (c = 7 kPa, φ = 21°)

Slope	Dry Condition	Low flood Level Condition	High flood Level Condition	Rapid drawdown
1:1	1.536	1.244	0.904	0.66
1:1.5	2.133	1.634	1.508	0.963
1:2	2.255	1.669	1.478	0.986

Table 3 Results of Stability Analysis of Soil Sample Type 2 (c = 10 kPa, φ = 14°)

Slope	Dry Condition	Low flood Level Condition	High flood Level Condition	Rapid drawdown
1:1	1.477	1.206	0.916	0.672
1:1.5	2.092	1.594	1.485	0.953
1:2	2.213	1.626	1.461	0.984

Table 4 Results of Stability Analysis of Soil Sample Type 3 (c = 20 kPa, φ = 12°)

Slope	Dry Condition	Low flood Level Condition	High flood Level Condition	Rapid drawdown
1:1	1.581	1.305	0.841	0.6
1:1.5	2.162	1.66	1.576	1.02
1:2	2.284	1.692	1.554	1.054

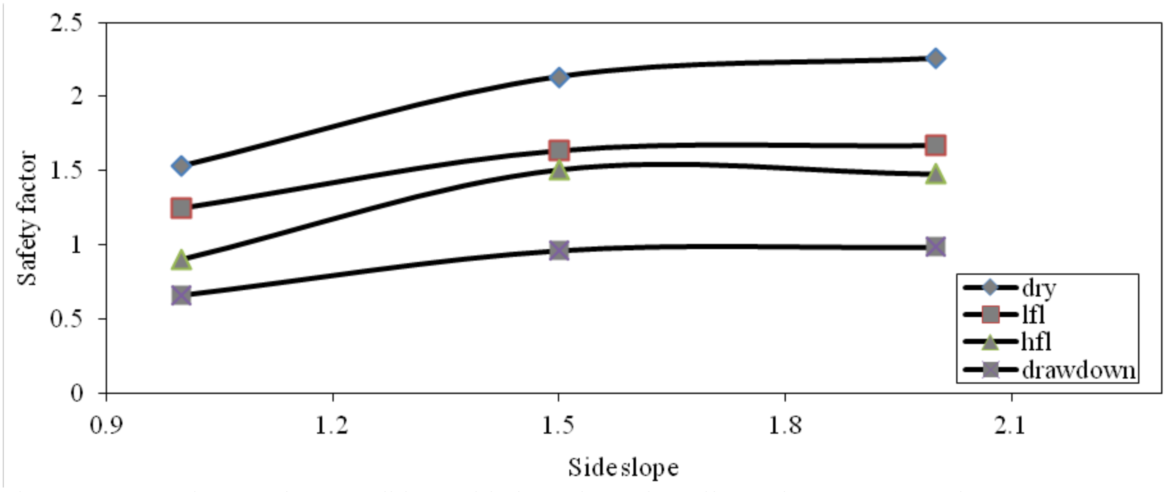

Figure 8: Comparison on three conditions with three slopes for soil sample Type 1 (c = 7 kPa, φ = 21°)

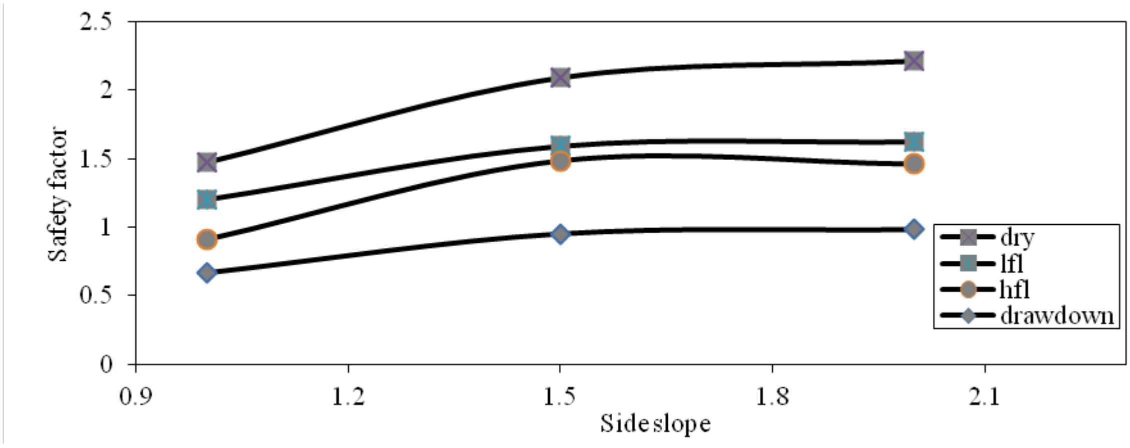

Figure 9: Comparison on three conditions with three slopes for soil sample type 2 (c = 10 kPa, φ = 14°)

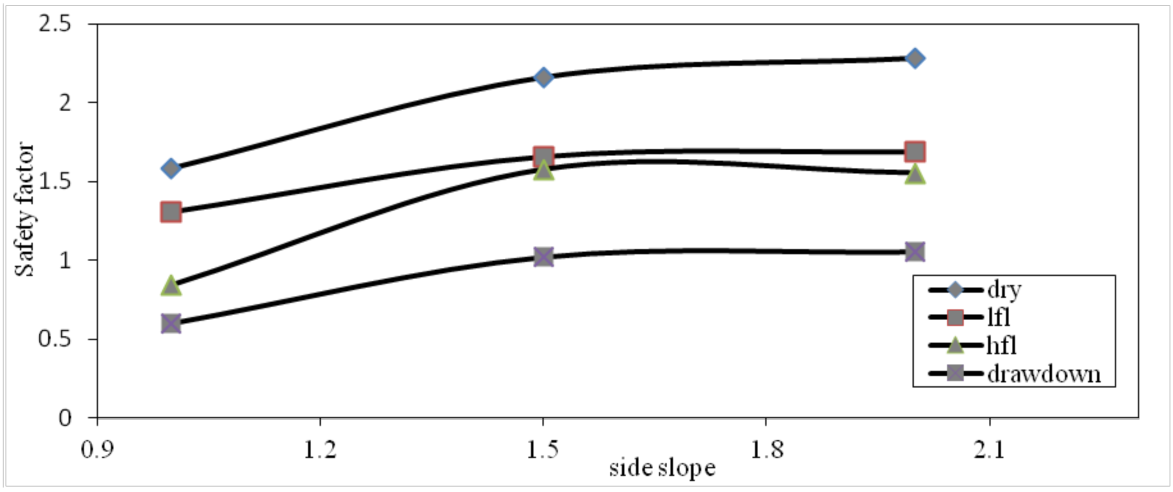

Figure 10: Comparison on three conditions with three slopes for soil sample Type 3 (c = 20 kPa, φ = 12°)

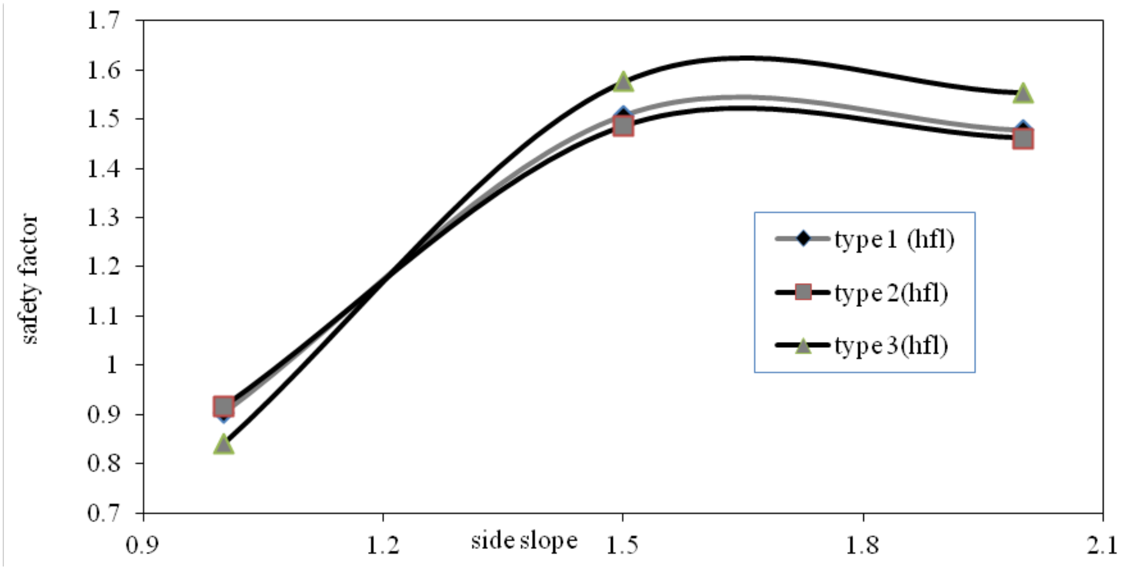

Figure 11: Variation of safety factor for different soil condition at high flood level condition.

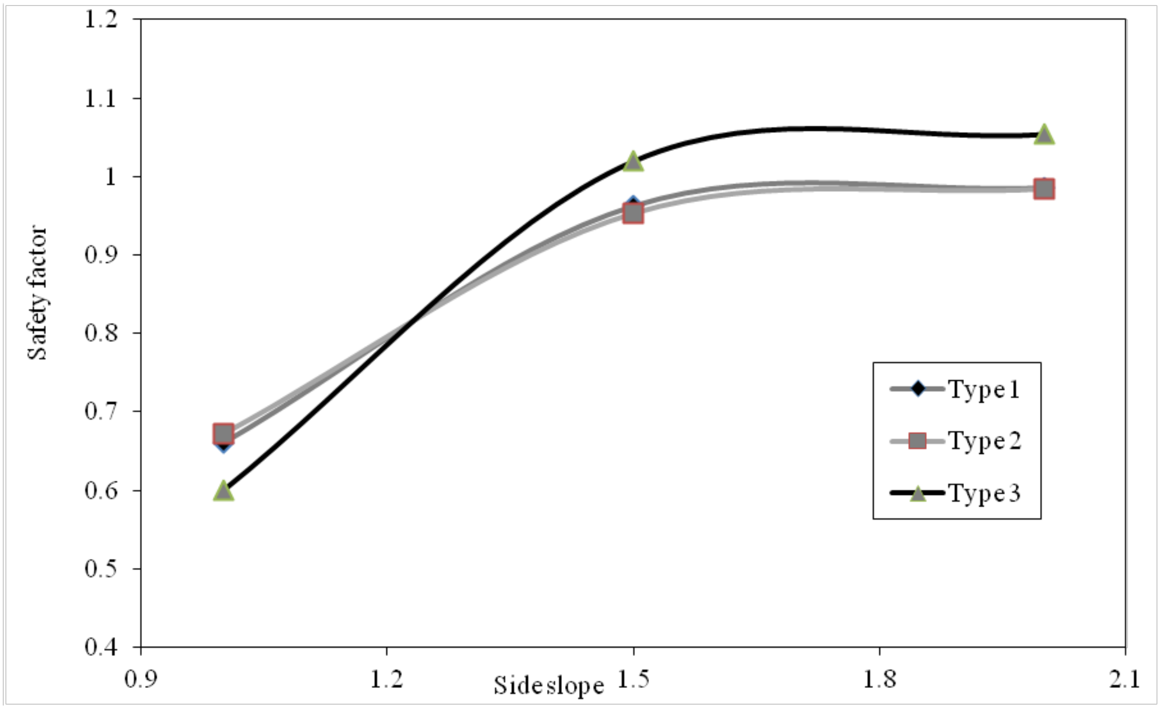

Figure 12: Variation of safety factor for different soil condition at rapid drawdown condition.

After analysis the safety factors for all types of soil at high flood level condition with slope 1:1 and at rapid drawdown for all three slopes have been found to be lower than the minimum recommended factor 1.2. So a typical design is needed to ensure the strength of soil. Revetment design is the most beneficial and affordable solution for Bangladesh. Using concrete block is considered to be efficient for Revetment design.

In the revetment design, concrete blocks are given in one or two layers, sometimes it extends to three, four or five layers either along with or without geotextile layer. It depends on the range of safety factor that is to be increased. From Stability analysis using STB2010, the safety factors for high flood level condition at slope 1:1 and rapid drawdown with three slopes for all categories of soil have been found to be below the minimum recommended value 1.2. So the revetment design has been performed here to raise the strength of soil. For the design, first a thin layer of geotextile has been given of 15 cm thickness. Then a layer is given with 45 cm * 45 cm * 45 cm concrete block. For the total layer including concrete block and geotextile, the unit weight would be the sum of $\gamma * h$ of both two materials. If the unit weight of concrete and geotextiles are 23.57 kN/m^3 (150 pcf) and 16 kN/m^3 (102 pcf) respectively then the total unit weight 13 kN/m^2 ($\gamma * h$) for 1 m strip would be counted for one layer in the design. When it is of two layers then the unit weight would be 23.5 kN/m^3 by adding extra unit weight of 2nd layer concrete block. The value of unit weight increases with the increase of concrete block layer adding 10.5 for each layer. Again it causes huge cost while increasing block layer one by one. As it costs much to layer the embankment overall with uniform concrete block, we can differ in placing the block layer depending on the satisfaction of safety factor. For sandy soil, four layers of concrete block have been placed at the bottom through toe up to the middle of the slope while two layers have been placed from middle up to the top of the embankment due to make the design economical. Table 5 shows the safety factor for different soil conditions at high flood level. For type 1 (c = 7 kPa, φ = 21°), four layers for high flood level condition have been placed at the bottom through toe up to the middle of the slope while two layers have been placed from middle up to the top of the embankment. For type 2 (c = 10 kPa, φ = 14°) four layers have been placed uniformly across the embankment for high flood level condition. For type 3 (c = 20 kPa, φ = 12°), four layers have been placed at the bottom through toe up to the middle of the slope while one layer has been placed from middle up to the top of the embankment for high flood level condition. Again for rapid drawdown condition, the distribution of block layers along with obtained safety factors have been given in table 6. The distribution of block layers is on the basis of making the design economical. Figure 13 shows the slope of an embankment indicating bottom and top. Table 7 shows the variation of safety factor for Type 1(c = 7 kPa, φ = 21°) depending on the distribution of concrete block layers. Safety factor has been found to be 1.198 for placing 4 layers at the bottom through toe up to the middle of the slope and 1 layer from the middle up to the top of the embankment. But it doesn't satisfy the condition. So 2 layers have been placed instead of 1 layer from the middle up to the top and the safety factor has been found to be 1.208. Table 8 shows the variation of safety factor for Type 2(c = 10 kPa, φ = 14°) depending on the distribution of concrete block layers. Safety factor has been found to be 1.204 for placing 4 layers uniformly across the embankment.

Table 9 shows the variation of safety factor for Type 3(c = 20 kPa, φ = 12°) depending on the distribution of concrete block layers. Safety factor has been found to be 1.288 for placing 4 layers at the bottom through toe up to the middle of the slope and 1 layer from the middle up to the top of the embankment which satisfies the condition. The variation of safety factor for different parametric soil at high flood level condition has been shown in figure 14 to 16.

Table 5 Results of Revetment Design at high flood Level condition.

Embankment Soil Properties	Factor of safety
c = 7 kPa, φ = 21°	1.208
c = 10 kPa, φ = 14°	1.204
c = 20 kPa, φ = 12°	1.288

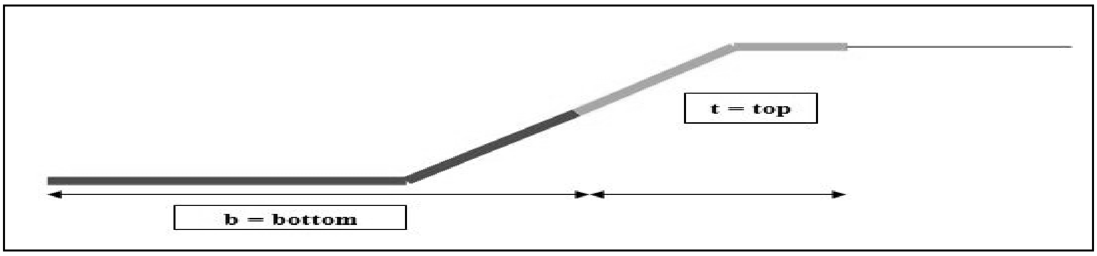

Figure 13: Slope of an embankment

Table 6 Results of Revetment Design at Rapid drawdown condition.

slope	c = 7 kPa, φ = 21°	c = 10 kPa, φ = 14°	c = 20 kPa, φ = 12°
1:1	1.201 (7b+1top)	1.200(7b+2top)	1.213 (6b+1top)
1:1.5	1.214 (3b+1top)	1.208 (3b+1top)	1.273(3b+1top)
1:2	1.276 (3b+1top)	1.268 (3b+1top)	1.213(1b+1top)

Table 7 Variation of safety factor of Type1(c = 7 kPa, φ = 21°) soil

Layer 1	safety factor	Layer 2	safety factor	Layer 3	safety factor	Layer 4	safety factor
1b + 1t	0.968	2b + 1t	1.043	3b + 1t	1.12	4b + 1t	1.198
1b + 2t	0.981	2b + 2t	1.055	3b + 2t	1.131	4b + 2t	1.208
1b + 3t	0.994	2b + 3t	1.067	3b + 3t	1.141	4b + 3t	1.218
1b + 4t	1.006	2b + 4t	1.078	3b + 4t	1.151	4b + 4t	1.227

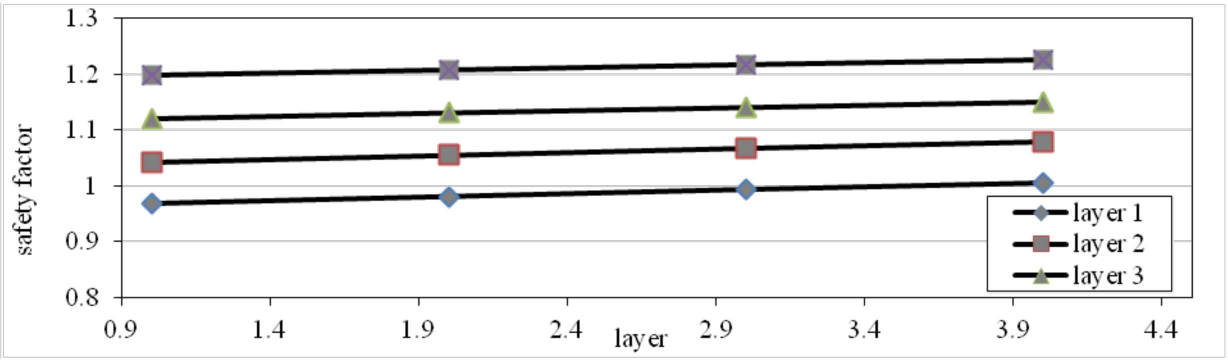

Figure 14: Safety factor vs. layer graph for Type1(c = 7 kPa, φ = 21°) soil.

Table 8 Variation of safety factor of Type2(c = 10 kPa, φ = 14°) soil

Layer 1	safety factor	Layer 2	safety factor	Layer 3	safety factor	Layer 4	safety factor
1b + 1t	0.951	2b + 1t	1.024	3b + 1t	1.098	4b + 1t	1.175
1b + 2t	0.964	2b + 2t	1.036	3b + 2t	1.109	4b + 2t	1.185
1b + 3t	0.976	2b + 3t	1.0477	3b + 3t	1.12	4b + 3t	1.195
1b + 4t	0.988	2b + 4t	1.058	3b + 4t	1.131	4b + 4t	1.204

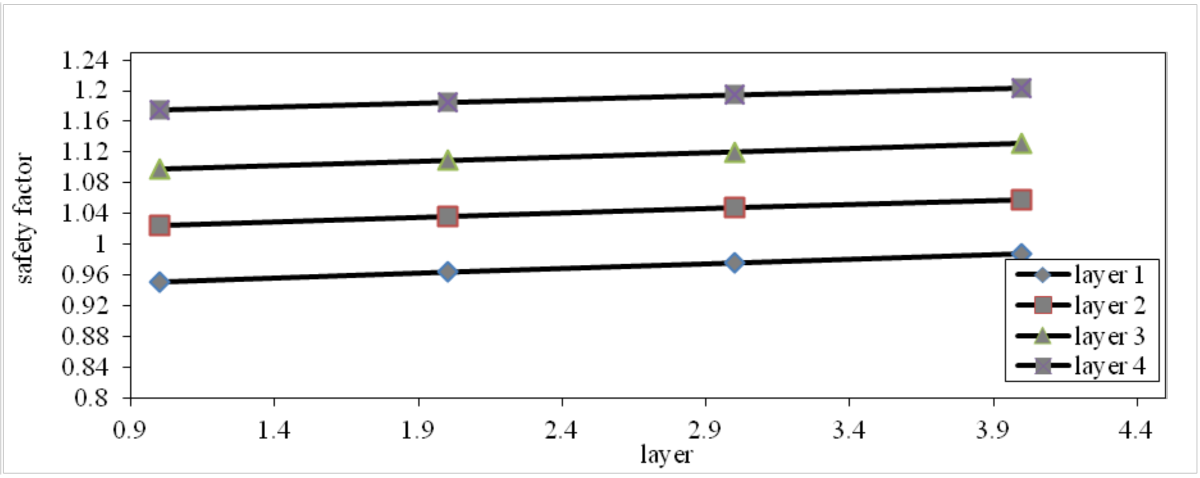

Figure 15: Safety factor vs. layer graph for Type2 (c = 10 kPa, φ = 14°) soil.

Table 9 Variation of safety factor of Type3(c = 20 kPa, φ = 12°) soil

Layer 1	safety factor	Layer 2	safety factor	Layer 3	safety factor	Layer 4	safety factor
1b + 1t	1.059	2b + 1t	1.134	3b + 1t	1.21	4b + 1t	1.288
1b + 2t	1.071	2b + 2t	1.144	3b + 2t	1.219	4b + 2t	1.296
1b + 3t	1.082	2b + 3t	1.154	3b + 3t	1.228	4b + 3t	1.304
1b + 4t	1.092	2b + 4t	1.164	3b + 4t	1.237	4b + 4t	1.312

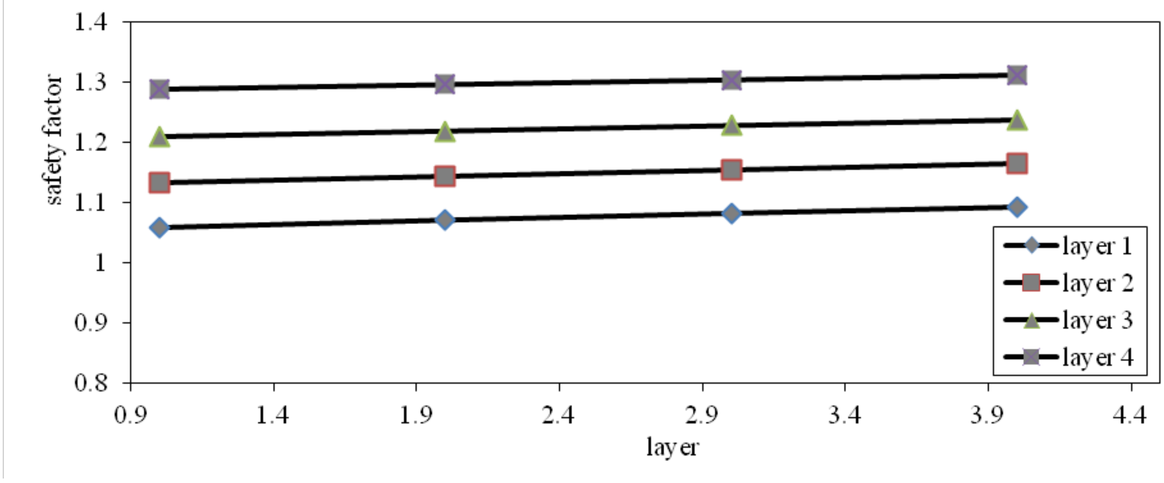

Figure 16: Safety factor vs. layer graph for Type 3 (c = 20 kPa, φ = 12°) soil.

After Revetment Design the conditions are satisfied for the soil samples at high flood level and rapid drawdown conditions. For all cases, the safety factors are above the recommended value (1.2). These values have ensured the shear strength of soil along with the protection of river embankment. The soil having better safety factors are assumed to be more protective from erosion. After the design, the number of layers at rapid drawdown has been found to be greater in quantity than high flood level condition. But in practice, rarely rapid drawdown condition is considered, so the number of layers used is minimum.

7. CONCLUSION

This research has been carried out to investigate the geotechnical characteristics of the embankment and presented results of more recent soil investigation along the embankment alignment. For this purpose embankment soil has been collected from Basuria in Sirajganj near the bank of Jamuna River. Also field bore logs has been done up to a depth of 30 m. Direct from the broken part of the embankment, the soil samples are collected by which the following laboratory tests: Grain Size Analysis, Compaction Test, Shear Test have been performed. The collected sample contains 63% sand, 35 % silt and 2% clay. According to the Unified Soil Classification System (USCS), the soil sample is SW – SM.

With a view to making a remolded sample to obtain the shear strength parameters of the collected disturbed sample of the embankment, a Standard Proctor Compaction test has been conducted in the laboratory. The remolded sample has been made with the corresponding water content of 95% peak value of the dry density at wet side. The optimum dry density has been found to be 17.45 kN/m^{3}. Due to obtain the shear strength parameters, Consolidated Undrained Shear test has been performed in the laboratory. The cohesion and angle of internal friction has been found to be 7 kPa and 21° respectively. For further parametric study, Shear strength parameters has been modified to be 10 kPa; 14° and 20 kPa; 12°. The parameters of the soil sample of the embankment have been found directly from the laboratory tests, while for the underlying soils the shear strength parameters have been obtained from the correlation with SPT – N value shown in the bore logs.

Based on the data of the present investigation, stability analysis of some critical sections of the embankment has been carried out. The stability analysis has been conducted using STB2010. The analysis depends on the soil parameters obtained during the construction of embankment. The analysis has been performed for soils at three conditions; dry, low flood level, high flood level and rapid drawdown with three different slopes; 1:1, 1:1.5 and 1:2. The values of cohesion and angle of internal friction obtained from the shear test have been used in STB2010. The maximum safety factor has been obtained 2.255 for soil at dry condition with a slope 1:2 while the minimum factor is 0.66 at rapid drawdown condition with 1:1 slope. As long as water increases, the soil becomes weakened for steepening slope. But at rapid drawdown condition safety factor has been found to be the least because of rapid reduction of external water level. So the soil would fail at any time as having lower shear strength to protest against erosion. It has been realized that, soil at dry condition has better strength to protect embankment from failure. Again the strength increases with flatter slope rather than steep slope. The main reason of the failure at high water level is considered to be water seepage into the soil while the reason is rapid reduction of external water level for rapid drawdown condition. For this type of soil, a design named Revetment Design with a thin geotextile layer and concrete block layer has been conducted to protect river embankment.

This design is affordable and suitable for Bangladesh. After Revetment Design a reasonable safety factor has been achieved for the soils at critical condition which ensures the protective strength of soil against failure.

REFERENCES

Alam, M.K., Hasan, A.K.M.S., Khan, M.R. “Geological Map of Bangladesh: Geological survey of Bangladesh”, Whitney, John W., 1990, United States Geological Survey.

Bhuiyan, M.A.H., Rakib, M.A., Takashi, Rahman, M.J.J. and Suzuki, Shigeyuki “Regulation of Brahmaputra-Jamuna River around Jamuna Bridge Site, Bangladesh: Geoenvironmental Impacts”. *J. Water Resource and Protection*, 2010, 2, 123-130 Doi:10.4236/jwarp.2010.22014 Published Online February 2010 (http://www.SciRP.org/journal/jwarp/).

Das, B.M. (1985). “Principles of Geotechnical Engineering”, PWS publisher.

Flaate, K. and Preber, T. (1974). “Stability of Road Embankments”, Canadian Geotechnical Journal, Vol. 11, No. 1, pp. 72 – 78.

Hoque, M.M. and M.A.B Siddique, 1995. Flood control projects in Bangladesh: Reasons for failure and Recommendations for improvement. Disasters, 19: 260-263. DOI: 10.1111/j.1467- 7717.1995.tb00344.x

Hossain, M.Z. and Sakai, T. “River Embankment and Bank Failure: A Study on Geotechnical Characteristics and Stability Analysis”, American Journal of Environmental Sciences 7 (2): 102-107, 2011

http://www.kennisbank-waterbouw.nl/DesignCodes/rockmanual/chapter%208.pdf

Morii, T. and H. Kunio, 1993. Finite element analysis of stress and stability of earth dams during reservoir filling. J. Fac. Agric., Tottori Univ., 29: 45-53.

M. Z. Hossain and T. Sakai. “Severity of Flood Embankments in Bangladesh and Its Remedial Approach”. Agricultural Engineering International: the CIGR Ejournal. Manuscript LW 08 004. Vol. X. May, 2008.

Thornton, Christopher I. and Kane, Richard (October 2007). “Revetment Design Considerations in Sheltered Water Wave Conditions”. Professional Development Series.

Verruijt, A., Delft University, 2010, a.verruijt@verruijt.net

CYCLIC AND MONOTONIC RESPONSE OF PADMA SAND AND SILT

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ABSTRACT

A series of stress controlled cyclic triaxial test has done at post consolidated 60 percent relative density with 100kPa effective confining pressure on sand and silt collected from the Padmariver bank (near proposed Padma bridge). Elsewhere strain controlled undrained monotonic triaxial test at same condition has been done. The excess pore water pressure come up quickly on silt than sand due to cyclic loading. Fine silt particles have low permeability than sand particles, so due to sudden loading loss of strength is higher for silt.

Keywords: *Cyclic loading, undrained monotonic loading, permeability, liquefaction, shear strength.*

1. INTRODUCTION

The loss of strength of saturated soil due to cyclic or static load of has been one of the most important and challenging problem in the field of Geotechnical Engineering. Nearly 85 percent of Bangladesh is underlain by deltaic and alluvial deposits of the Ganges, Brahmaputra and Meghna river systems. The delta deposits are sediments that are deposited on the active delta, which is defined as the area south of the Ganges River and mostly west of the Meghna estuary (Alam, et al. 1990). In rainy season sandy soil deposit occurs and in winter season silty soil deposit occurs for the change of stream flow rate with season.

Until now it is not clear that which one is more susceptible to failure due to static or cyclic. Sand shows more strain softening or liquefiable than silty sand or silt (Sing, 1994; Amini and Qi, 2000) on the other hand, reverse behaviour is explained by Polito and Martin II (2001) and Dash and Sitharam (2011). In this regard our target to justify the behavior of sand and silt.

2. EXPERIMENTAL PROGRAM

2.1 Materials Used

Fine sand and silt was collected from sandbars of Padma River, Mawa, Munshiganj, Bangladesh, near proposed Padma Bridge site. Fine sand and silt both was oven dried and then sieved through 75 µm sieve to obtain the clean sand and silt. The grain size distribution of clean sand, silt and sand-silt mixtures are presented in Figure 1. The silts are found non-plastic. Index properties of sand and silt are shown in Table 1. Fine sand and silt specimen were viewed under Scanning Electron Microscope (SEM) to see the shape of particles. Figure 2 (a) shows the image of fine sand with 50 times magnification and Figure 2 (b) shows the image of silt with 150 times magnification. From these two images from SEM it is clearly seen that the fine sand and silt particles are angular and rough. That means fine sand and silt; both are same granular material with different particle sizes. To estimate mica content in the fine sand and nonplastic silt, X-RD test have done, because Hight et al. (1999) reported that increase of Mica content decreases the undrained shear strength of sand. It is found that mica exist 0% in sand and 4% in silt (this percent is based on atomic weight). And it should be noted that about 8 percent of clay mineral illite is present in the silt. However it shows non-plastic behavior. The presence of minerals in sand and silt is shown in Table 2. Maximum and minimum index densities of sand and silt sample were determined and shown in Table 1. Maximum density was determined by modified proctor test and minimum density was determined by free fall method. Cyclic triaxial tests were performed on sand and silt specimens with various cyclic stress ratios and monotonic (or static) triaxial test were at 0.05% axial strain per minute. The experimental program is shown in Table 3.

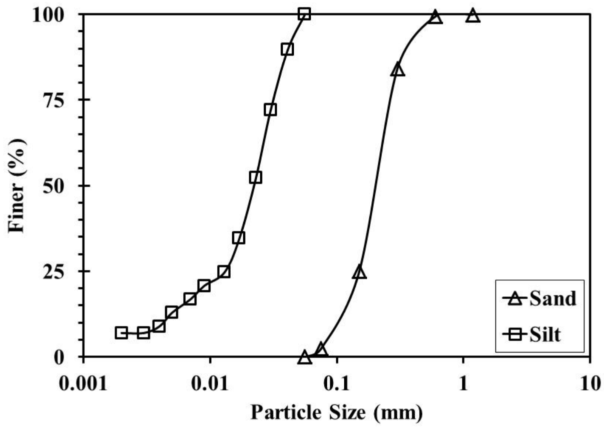


Figure 1: Grain size of the samples.

Table 1: Index properties of sand and silt used in the study.

Soil type	Sand	Silt
USCS classification symbol	SP	ML
Mean grain size D_{50} (mm)	0.203	0.022
Uniformity coefficient (C_u)	2.25	5.82
Coefficient of gradation (C_z)	1.17	2.15
Specific gravity (G_s)	2.69	2.72
Maximum dry density (kN/m^3)	17.51	18.40
Minimum dry density (kN/m^3)	12.17	8.20

Table 2: Quantitative X-RD Test Result.

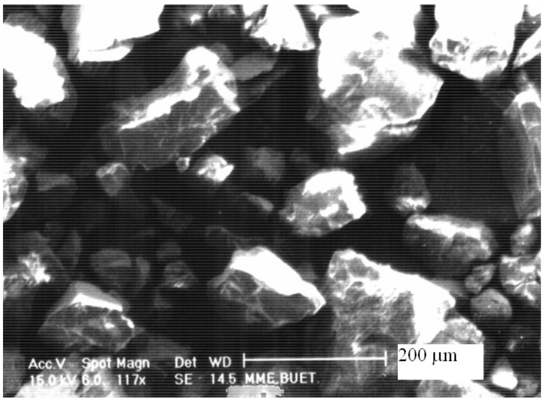
Soil Type	Compound Name		Percent (%) (By Atomic Weight)	Formula
Sand	Quartz		81.10	SiO_2
	Albite		11.28	$Na(AlSi_3O_8)$
	Fledspar		7.62	$K_{0.5}Na_{0.5}AlSi_3O_8$
Silt	Quartz		66.62	SiO_2
	Chlorite		5.72	$(Mg, Al)_6(Si, Al)_4O_{10}(OH)_8$
	Albite		8.24	$Na(AlSi_3o_8)$
	Clay Mineral	Illite	8.11	$(K, H_3O)AlSiAlO_{10}(OH)_2$
	Mica	Muscovite	4.35	$Kal_2(AlSi_3O_{10})(OH)_2$
	Vermiculite		4.89	$Mg_{3.4}Si_{2.85}Al_{1.1}O_{10}(OH)_2(H_2O)_{3.7}$
	Megnesium iron silicate		2.07	$Mg_{0.8}Fe_{0.2}(SiF_6)(H_2O)_6$

Table 3: Experimental Program.

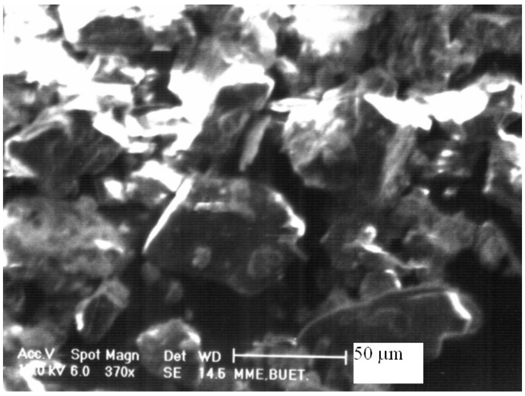
Soil type	Isotropic consolidation (kPa)	Cyclic triaxial test		Static triaxial test	
		D_r (%)	*CSR* (*f*=1Hz)	D_r (%)	Axial strain rate
Sand	100	60	0.08-0.2	30, 36, 40, 60	0.05%/min
Silt	100	60	0.08-0.2	30, 60, 73, 76	0.05%/min

2.2 Specimen Preparation

Soil specimens used in this study were of 71 mm in diameter and 142 mm in height. The specimens were formed by using wet tamping method in a split mold. The inner-diameter of the mold is 71 mm and height 142 mm. The dry soil is mixed with 10 percent water and then compacted in several equal layers by a hammer that delivers some blows to each layer to achieve the target relative density (Moist tamping method). Number of layers and number of blows per layer were determined by trial to achieve target relative density. The hammer weighs 1 kg, and had a drop of 6 inches. In order to obtain a uniform density throughout the specimen, the compaction method of specimen preparation suggested by Ladd (1978) was used. After that the specimen is placed in the platform or cell chamber and then griped with a membrane. Two o-rings are used in top and bottom of the specimen to resist the lick age of water in cell chamber to soil. Later the cell chamber is griped with high pressure to resist the lick age of water filled.



(a)



(b)

Figure 2: Electro microscopic view of (a) sand and (b) silt.

2.3 Saturation, Consolidation and Static or Cyclic Loading

While the preparation of the specimen was completed and fixed under the actuator in a water filled chamber, initial saturation of the specimen was done by passing carbon dioxide about one hour through the specimen. After that the distilled water was passed through the specimen by gravity pressure of 5 kPa for 3 to 5 hours. At the end of this process the machine was switched on. The machine is capable of applying sufficient back pressure till it was ensured that the Skempton's B parameter equal to 95%. The specimens were then isotropically consolidated to a desired effective confining stress. The duration for the process of consolidation was varied from about 2 hours (for clean sands) to about 3 hours (for pure silt). All relative densities reported here are post consolidation relative densities. After consolidation cyclic load was applied at various cyclic stress ratios (cyclic test) or monotonic load at 0.05% axial strain per minute. At cyclic phase, it is needed to make a vacuum space of 1/2 inch thick in the cell chamber which pressure will be equal to cell pressure. Then it goes for cyclic loading as the Cyclic Stress Ratio (CSR) was imparted. At monotonic test the test continues until imparted axial strain is achieved by the sample. The cyclic and static triaxial test is done by the same machine (as in Figure 3) but after the consolidation the procedures are different.

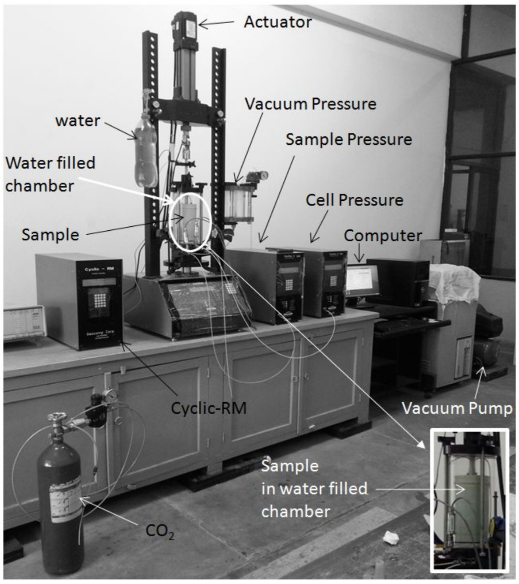
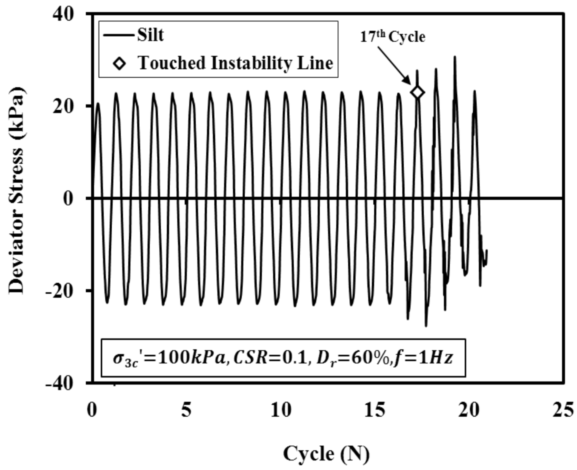


Figure 3: Cyclic and Static Triaxial test apparatus.

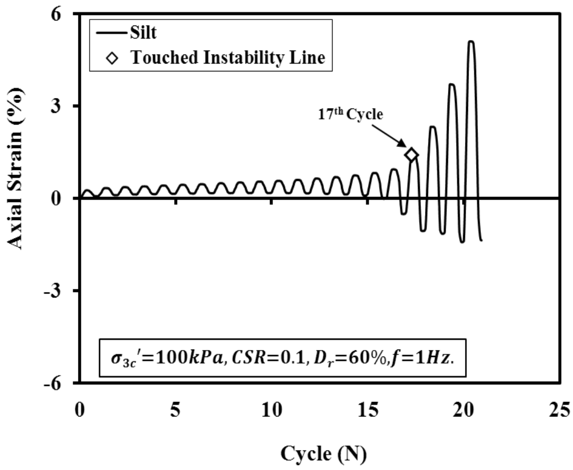
3. TEST RESULT

3.1 Cyclic Triaxial Test

The results of cyclic triaxial test on sand sample with 60 percent relative density, 100 kPa effective confining pressure, 0.1 Cyclic Stress Ratio (CSR) and 1 Hz Frequency (f) is exhibited in Figure 3, where deviator stress versus cycles (N) graph is shown in Figure 3 (a). Axial strain verses cycle graph is in Figure 3 (b), where the constant load applied to the specimen till ±3% axial strain was developed. It is apparently noted that after 17th cycles the peak axial strains increase drastically with increase of cycles than that of prior cycles. Excess pore pressure response of silt is shown in Figure 3 (c). At 20th cycle it shows 87 kPa excess pore water occurred in the sample. Another thing is that, the rate of generation of excess pore water pressure is higher after 17th cycle then that of before, as in axial strain. Figure 3 (d) is exhibiting q = (σ1 - σ3)/2 verses p' = (σ1'+σ3')/2 graph, where the line becomes scribble after 17th cycle because of the deviator stresses could not achieve the prescribed amplitudes. The reason of this type of behaviour is that at 17th cycle the sample reaches the Instability Line (IL) (line through qpeak point). The employed actuator is a pneumatic device that is operated by air pressure. It could be another reason why the cyclic load of 1 Hz frequency in Figure 3 (a) could not achieve the prescribed amplitude when IL reaches. Use of lower frequency may solve the problem. Similar type of behavior was observed for sand specimen.



(a)



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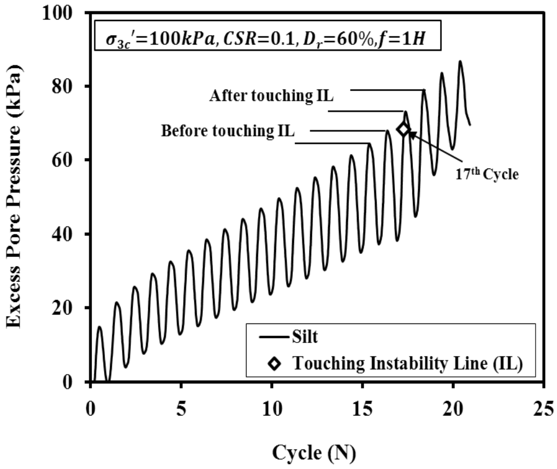
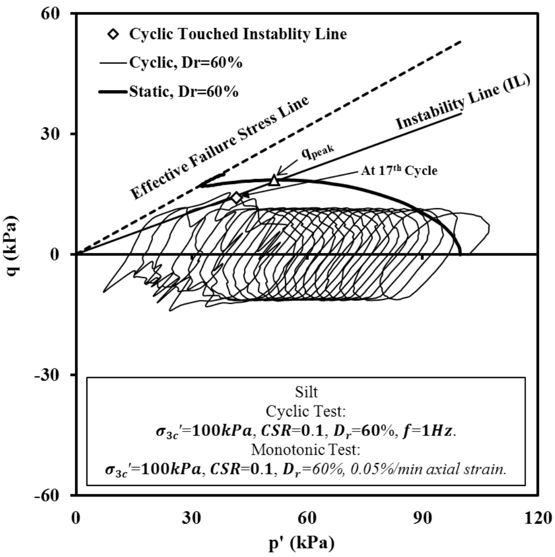


(c)



(d)

Figure 4: (a) Deviator stress verses cycles of loading, (b) axial strain verses cycles of loading, (c) excess pore water pressure response and (d) effective stress path.

To define the Cyclic Shear Strength (*CSS*), minimum three cyclic tests were done on each sample. Cyclic Stress Ratio (*CSR*) at 15th cycle for initial liquefaction ($\sigma_1 = \sigma_3$) or ±3 % double amplitude is used for *CSS* calculation (see Figure 4). Cyclic Shear Strength,

$$\sigma_c = 2\sigma_{3c}^{'} CRR \tag{1}$$

Where, σc = cyclic shear strength (kPa), σ3c' = effective confining pressure (kPa), Cyclic Resistance Ratio (CRR) at 15th cycle. Here the CSS of silt at 100 kPa effective confining pressure and 60 percent relative density is 22 kPa, calculated.

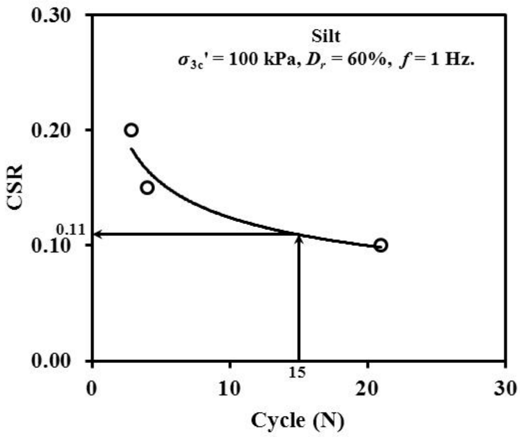
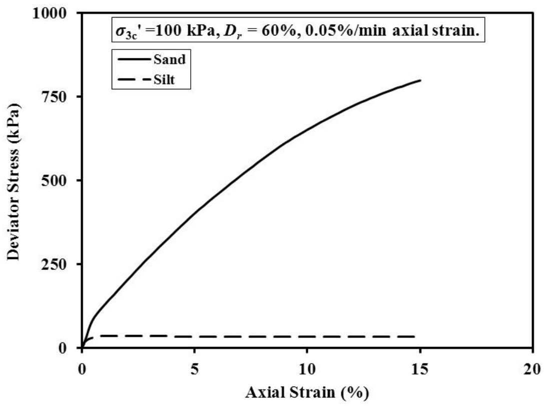


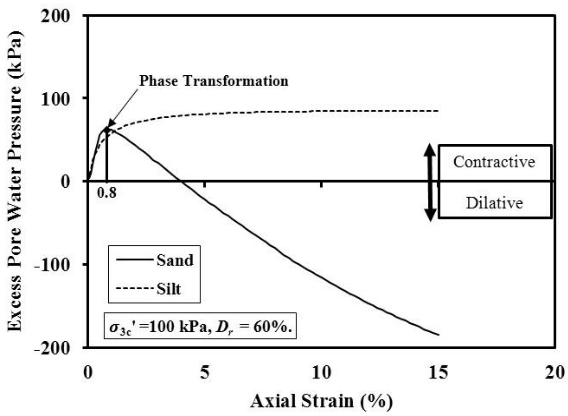
Figure 5: Cyclic stress ratio verses cycle graph.

3.2 Static Triaxial Test

The results of isotropic consolidated undrianed static test are presented in Figure 6. Deviator stress verses axial strain graph is shown in Figure 6 (a) where it is apparently noted that the specimen with 60% percent relative density, sand exhibited dilative behavior (Ishihara, 1996) but silt exhibited contractive behavior for the same relative density (60%). The silt specimen has reached a steady-state line with the deviator stress of 37 kPa and 1.2% axial strain. The Static Shear Strength (*SSS*) or undrained peak shear strength is the half of the peak deviator stress within 15 percent axial strain. Here the *SSS* of sand and silt at 100 kPa effective confining pressure and 60 percent relative were 400 kPa and 18.5 kPa respectively. The excess pore pressure response of silt is presented in Figure 6 (b). Initially for sand specimen the excess pore water pressure increased till 0.8 percent axial strain and achieved of 62 kPa which is denoted as contractive behavior (Rees, 2010). Later the dilative behavior exhibited due to decreasing excess pore water pressure (Rees, 2010). The point of change of contractive to dilative is denoted as phase transformation point. Figure 6 (c) shows the effective stress path of tested sand and silt specimen. Here the p' decreased for silt but for sand p' initially deceased after that it deviated upwards, this transition point is also called phase transformation point. The prior phase transformation point and this one are the same one. The deviation of p' is due to the deviation of excess pore water pressure from increasing to decreasing. The angle of effective failure stress line is 31^{o}. Similar behavior was found for all specimens.



(a)



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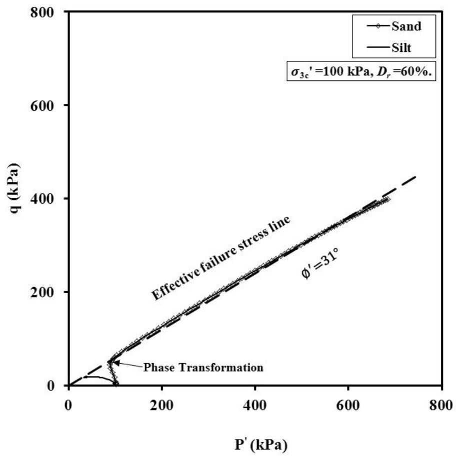

(c)

Figure 6: (a) Stress-strain response, (b) excess pore pressure response, (c) effective stress path of sand at D_r=60%.

4. DISCUSSION

4.1 Static Triaxial Test

To explain the behavior of sand and silt at 60 percent relative density a provable soil structure is shown in Figure 7. The permeability test result of sand and silt by Fall Head method at 60% relative density on is shown in Figure 8. The permeability of sand is higher than that of silt. Because the voids of among the sand particles are greater than that of the silt particles at the 60 percent relative are shown in Figure 7 (i). Figure 7 (ii) shows the applied shear stress and normal effective stress in this stable particles structure. In the shearing phase inner slip occur among the sand particles (Figure 7 (ii) (a)) which procreate inner voids. As it is undrained triaxial test whereupon a large amount of suction pressure (excess negative pore water pressure) develop (as in Figure 5 (b)) in the voids which increases effective mean pressure. Consequently the peak shear stress is higher ($\sigma = \sigma'_{3c} + (\pm u)$) for sand. In silt at the shearing phase the silt particles got more contract to each other (Figure 7 (ii) (b)), as it is an undrianed test so huge excess pore water pressure develops (Figure 6 (b)). Initially the excess pore water pressure increased till 0.8% axial strainin sand (Figure 6 (b)). Because, the sand particles had contracted before 0.8% axial strain develop (Figure 6 (b)).

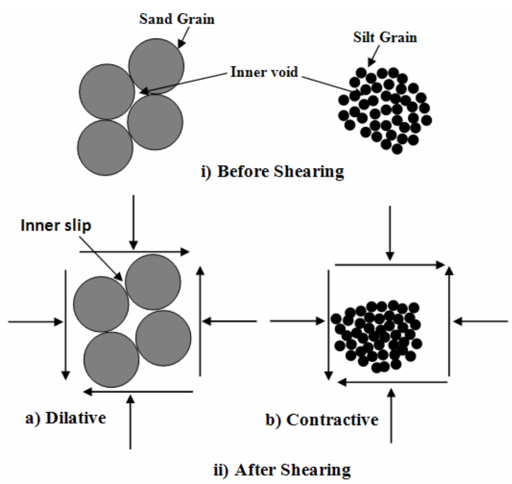

Figure 7: Sand and silt's structures i) before shearing and ii) after shearing.

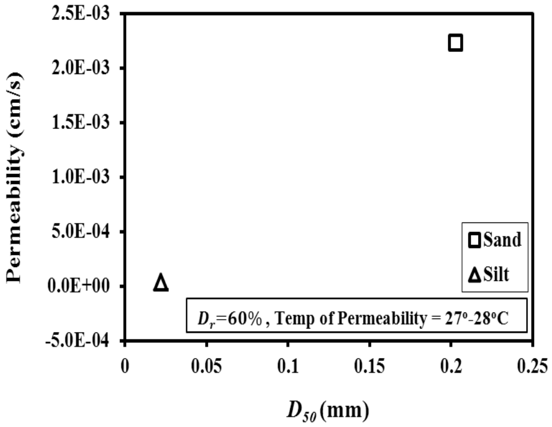

Figure 8: Permeability test on sand and silt at *D_r=60%*.

4.1.1 *Effect of relative density*

The undrained peak shear strength verses relative density graph is shone in Figure 9. It can be seen that at sand the peak shear strength is decreasing with decrease of relative density at 100 kPa effective confining pressure. Here after the consolidation relative density increased 0.5 to 2 percent for all relative densities of sands' specimen. In silt the peak shear strength is decreasing with decrease of relative density till 60 percent relative density, for further decrement of relative density the undrained peak shear strength increasing. The reason of this type of behavior is that at 30 percent relative density the relative density becomes greater than 60 percent relative density after consolidation at 100 kPa. Free fall method was used to determine the minimum dry density, where we got that 30% relative density is equal to the minimum density in water method. So, for much confining pressure (100 kPa) the specimen become denser than 60% relative density which is the reason of much peak shear strength at 30% relative density. For author silts' relative densities after consolidation relative densities increase near about 1%.

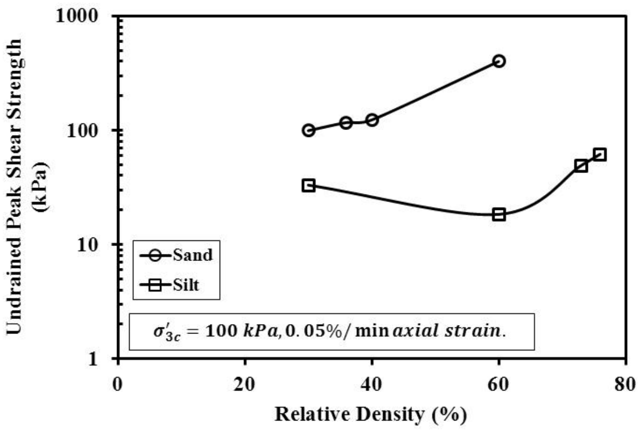

Figure 9: Undrained peak shear strength verses relative density.

4.1.2 *Effect of mica mineral in silt*

To justify the effect of mica mineral in silt, 1 g of mica (passing through 75 µm sieve) was added with 1000g silt. After mixing of silt and extra 1 g mica (0.1% mica, by weight), a specimen of 73 percent relative density was tested at 100 kPa isotropic effective pressures. The test result is shown in Figure 10, where the silt with 0.1% mica specimen has lost 34 percent of deviator stresses. That means mica flings the silt to lose the undrained shear strength.

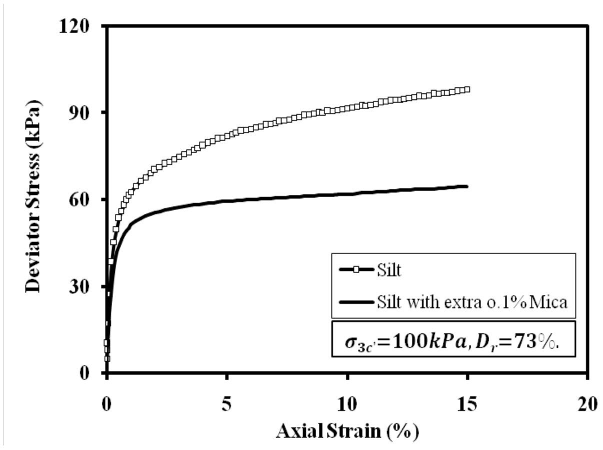

Figure 10: Effect of mica in silt.

4.2 Cyclic Triaxial Test

Due to sudden cyclic loading the developed excess pore water pressure inside void does not get time to dissipate, so the excess pore water pressure increases (Figure 11). In Figure 11 it can be seen that the increment rate of excess pore water pressure is faster for silt sample than sand (Karim and Alam, 2013). Fine silt particles have low permeability than sand particles. So, in cyclic loading silt dissipates less excess pore pressure than sand. For this reason the loss of strength is higher for silt. That means silt is more susceptible to liquefaction than sand. Figure 12 is exhibited the CSR verses cycles graph. Also for the reason the CSS of sand (34 kPa) is greater than that of the silt (22 kPa) as in Figure 12. Consequently silt is more susceptible to liquefy than sand.

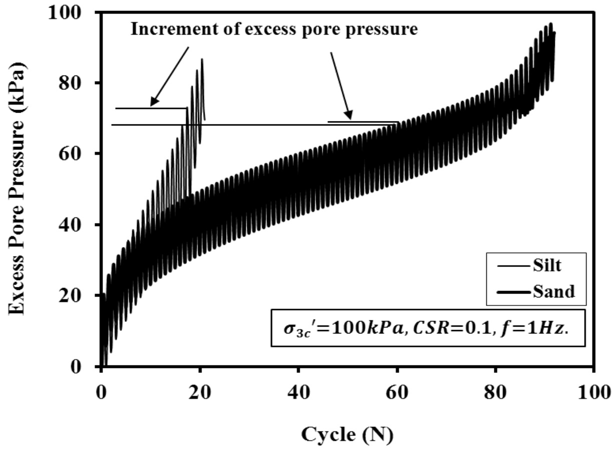

Figure 11: Excess pore water pressure ratio verses cycles of sand and silt.

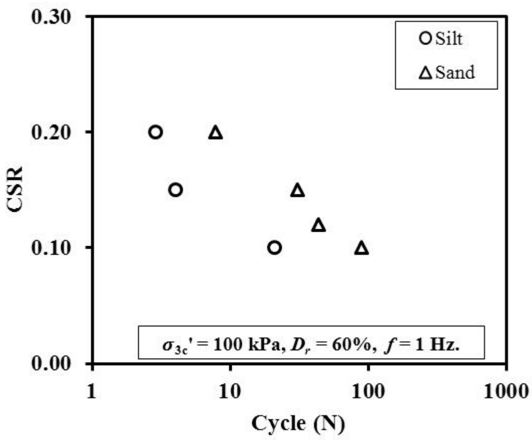

Figure 12: Cyclic stress ratio verses cycle of sand and silt.

5. CONCLUSIONS

1) In static undrianed test, sand exhibited dilative bahvior while silt showed contractive behaviour at 60% relative density.
2) The cyclic shear strength and undrained peak shear strength of sand is greater than that of the silt, because the permeability of sand is higher.
3) Increase of Mica content decreases the undrained shear strength of silt.

ACKNOWLEDGEMENTS

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REFERENCES

Amini, A., and Qi, G. Z., (2000) "Liquefaction Testing of Stratified Silty Sands." Journal of Geotechnical Geoenviromental Engineering, Proc. ASCE, Vol. 126(3), 208-217.

Alam, M. K., Hasan, A. S., Khan, M. R., and Whitney, J. W. (1990). "Geological Map of Bangladesh." Government of the Peoples Republic of Bangladesh Ministry of Energy and Resources Geological Survey of Bangladesh. Bangladesh.

Dash, H. K., and Sitharam, T. G. (2011). "Undrained Cyclic and Monotonic Strength of Sand-silt Mixtures." Geotech Geol Eng, Springer, Vol. 29, 555–570. DOI: 10.1007/s10706-011-9403-3

Hight, D. W., Georgiannou, V.N., Matin, P.L. and Mundegar, A.K. (1999). "Flow slides in micaceous sands", Proc. International Symposium on Problematic Soils. Sendai, Japan, Vol. 2, 945-958.

Karim, M. E. and Alam, M. J. (2013). "Response of Sand-Silt Mixture Under Cyclic and Undrained Monotonic Loading." 8th CUTSE International Conference, Curtin University Sarawak, Malaysia.

Ladd, R. S. (1978). "Preparing Test Specimens Using Undercompaction." Geotech. Testing Journal, ASTM, Vol. 1, (1), 16-23. DOI: 0.1520/GTJ10364J

Polito, C. P., and Martin II, J. R. (2001). "Effects of Nonplastic Fines on the Liquefaction Resistance of Sands." Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 127(5), 408-415, DOI:10.1061/(ASCE)1090-0241(2001)127:5(408)

Rees, S. D. (2010). "Effect of Fines on the Undrained Behavior of Christchurch Sandy Soils." Ph.D. Thesis Submission to University of Canterbury Christchurch, New Zealand.

Sing, S., (1994) "Liquefaction Characteristic of Silt." Ground Failure Under Seismic Condition Geotechnical Special publication, ASCE, No. 44, 105-116.

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EFFECT ON PERMEABILITY AND SHEAR STRENGTH WITH THE VARIATION OF GRAIN SIZE OF SAND

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ABSTRACT

Permeability and shear strength are very important properties of a soil which are affected by grain size of soil particles. For the improvement of soft soil sometimes sand pile are used. The main objective is to investigate which type of sand is more suitable for sand pile economically. In this project, an attempt is made to relate permeability and frictional angle with the properties of grain size of soil such as effective size (D10), mean particle size etc. The properties of grain size can be found out from gradation curve. The results of coefficient of permeability vary from 1× 10-3 to 15× 10-3. The permeability increased significantly with the increase of mean particle size and effective size of sand. Frictional angle increased with the increase of permeability of soil but the variation is very insignificant. Shear Strength also increased with the increase of mean particle size but the variation is also insignificant in this case. As the changes of value of frictional angle is negligible with the changes of size of sand from fine sand to coarse sand, local sand may be used with certain percentages of Kushtia sand in lieu of percentages of Sylhet sand in case of designing sand pile economically.

Keywords: *Permeability, Sand Drain, Shear Strength, Frictional Angle, Local Sand*

1. INTRODUCTION

Most of the people are now willing to migrate to urban area for an improved life standard. This is making urban area more demanding, resulting in non-availability of land for building construction. A building of heavy weight requires heavy foundation, which warrants costly foundation system, if the soil is of low bearing capacity like Khulna. For the improvement of soft soil sometimes sand pile is used. The effectiveness of this pile depends mainly on the pore water dissipation rate, which may be variable with the size of granular fills (sand).In this project works, it will be under investigation that which type of sand is more suitable for sand pile economically. Granular materials are preferred for structural fill because they are strong, drain water rapidly, and settle relatively little. An important application of granular materials is backfill in mechanically stabilized earth (MSE) walls and reinforced soil (RS) slopes. For these applications, the friction angle of the sand generally is the most important property. In geotechnical engineering, the porous medium is soils and the fluid is water at ambient temperature. Generally, coarser the soil grains, larger the voids and larger the permeability.

Sand piling is a cost-effective method of ground improvement by compaction which is commonly used to improve soft seabed soils prior to land reclamation works. The benefits of gravel drains are densification of surrounding non-cohesive soil, dissipation of excess pore water pressure and re-distribution of earthquake-induced or pre-existing stresses. The rate of drainage of water from soil depends on the permeability. Volume change under load takes place quickly in sands and gravel. Piles composed of particles of same size do not produce a stress depression, while piles composed of particles with varying size produce stress depression. The sand pile required high strength and high permeability sand.

2. METHODOLOGY

2.1 General

The materials and method was divided into four steps. In the first step, three different sand were collected and reconstituted sands were made by the mixture of collected sands in different proportions.

The second step included grain size distribution to determine D_{10}, D_{30} and D_{60} from particle size distribution curve of the samples.

The third step consists of compaction test to determine optimum moisture content for wet samples in direct shear test.

Lastly, permeability and direct shear test were done for determining coefficient of permeability and angle of friction of the samples respectively.

2.2 Preparation of Samples

2.2.1 Introduction

Three types of sand were collected. They are local sand, kushtia sand and sylhet sand.

2.2.2 Mixture of Local and Kushtia Sand

The local and kushtia sand were first oven dried. Then the samples were prepared by mixing them in different proportion like 75% local and 25% kushtia , 50% local and 50% kushtia and 25% local and 75% kushtia sand.

2.2.3 Mixture of Local and Sylhet Sand

The local and sylhet sand were first oven dried. Then the samples were prepared by mixing them in different proportion like 75% local and 25% sylhet, 50% local and 50% sylhet and 25% local and 75% sylhet sand.

2.2.4 Mixture of Kushtia and Sylhet Sand

The kushtia and sylhet sand were first oven dried. Then the samples were prepared by mixing them in different proportion like 75% kushtia and 25% sylhet, 50% kushtia and 50% sylhet and 25% kushtia and 75% sylhet sand.

2.3 Determination of Coefficient of Permeability

The four most common laboratory methods for determining the coefficient of permeability of soils are the following:

1) Constant-head test
2) Falling head test
3) Indirect determination from consolidation test
4) Indirect determination by horizontal capillary test

Constant-Head Test

The constant head test is suitable for more permeable granular materials. Water flows from the overhead tank consisting of three tubes: the inlet tube, the overflow tube and the outlet tube. The soil specimen is placed inside a cylindrical mold, and the constant head loss, h, of water flowing through the soil is maintained by adjusting the supply. The outflow water is collected in a measuring cylinder, the duration of the collection period is noted. From Darcy's law, the total quantity of flow Q in time t can be given by

$$Q = qt = kiAt$$

Where A= area of cross section of specimen

But $i = h/L$

where L is the length of specimen, and so

$$Q = k\,(h/L)\,At$$

Rearrenging this gives

$$k = QL/hAt \qquad (1)$$

The values of Q,L, H, A, t can be determined from the test, and then the coefficient of permeability k for a soil can be calculated from Eq.(1) (B.M.Das,Advance)

2.4 Compaction Test

The laboratory test generally used to obtain the maximum dry unit weight of compaction and the optimum moisture content is called the Proctor compaction test (Proctor, 1933). The procedure for conducting this type of test is described in the following section:

Standard Proctor Test

In the Proctor test, the soil is compacted in a mold that has a volume of 943.3 cm3. The diameter of the mold is 101.6 mm. During the laboratory test, the mold is attached to a base plate at the bottom and to an extension at the top. The soil is mixed with varying amounts of water and then compacted in three equal layers by a hammer that delivers 25 blows to each layer. The hammer weighs 24.4 N (mass: 2.5 kg), and has a drop of 304.8 mm. For each test, the moist unit weight of compaction can be calculated as

$\gamma = W/ V(m)$

where

W = weight of the compacted soil in the mold
V (m) = volume of the mold =943.3 cm3
For each test, the moisture content of the compacted soil is determined in the laboratory. With known moisture content, the dry unit weight γd can be calculated as

$\gamma d = \gamma / 1+(w/100)$

The values of γd determined can be plotted against the corresponding moisture contents to obtain the maximum dry unit weight and the optimum moisture content for the soil. (Punmia, B.C. 1977)

2.5 Direct Shear Test

The shear strength parameters for a particular soil can be determined by means of laboratory tests on specimens taken from representatives samples of the in-situ soil.

Basically, the equipment for this test consists of a metal shear box into which the soil specimen is placed. The specimen can be square or circular in plan, about 3 to 4 in2 (19.35 to 25.80 cm2) in area and 1 in (25.4mm) height. The box is split horizontally into two halves. The sample is between two porous stones, which are toothed or serrated to minimize the slippage at the interface between soil and shear box and to improve the transfer of the shear load to the soil. The porous stones are also used to drain water from saturated samples. The screws are used to adjust the spacing between the upper and lower parts of the shear box. 'Two mounting pins maintain the position of these two parts during the sample fabrication and are removed before the beginning of the shear phase. The base is fixed to the loading frame and occasionally contains water when the soil sample is to remain saturated. The normal load is applied to the soil sample through a ball bearing and a rigid cap. The lateral load is applied to the upper part through the swan neck.

Normal force on the specimen is applied from top of the shear box by dead weights. The normal stress on the specimens obtained by the application of dead weights can be as high as 150lb/in2 (1035 kN/m2). Shear force is applied on the side of the top half of the box to cause failure in the soil specimen. During the test, the shear displacement of the top half of the box and the change in specimen thickness are recorded by the use of horizontal and vertical dial gauges. (B.M. Das, Advance).

3. ILLUSTRATIONS

3.1 Tables

Table 1: Results of Coefficient of Permeability obtained from Constant Head Test

Sample	Coefficient of Permeability (10^{-3} cm/sec)
100% local sand	1.09
100% kushtia sand	2.67

100% sylhet sand	14.7
75% kushtia sand & 25% local sand	1.74
50% kushtia sand & 50% local sand	1.44
25% kushtia sand & 75% local sand	1.31
25% local sand & 75% sylhet sand	9
50% local sand & 50% sylhet sand	2.77
75% local sand & 25% sylhet sand	1.31
75% kushtia sand & 25% sylhet sand	3.05
50% kushtia sand & 50% sylhet sand	4.72
25% kushtia & 75% sylhet sand	12.1

Table 2: Calculations of Mean Particle Size (D_{50}), Effective Size (D_{10}) and C_u

Sample	D_{50} (mm)	D_{10}(mm)	C_u
100% local sand	0.12	0.078	1.82
100% kushtia sand	0.245	0.13	2
100% sylhet sand	0.53	0.18	3.88
75% kushtia sand & 25% local sand	0.2	0.09	2.67
50% kushtia sand & 50% local sand	0.173	0.087	2.13
25% kushtia sand & 75% local sand	0.16	0.084	2.02
25% local sand & 75% sylhet sand	0.44	0.11	3.5
50% local sand & 50% sylhet sand	0.249	0.09	3.33
75% local sand & 25% sylhet sand	0.175	0.085	2.35
75% kushtia sand & 25% sylhet sand	0.449	0.16	3.75
50% kushtia sand & 50% sylhet sand	0.39	0.14	3.57
25% kushtia & 75% sylhet sand	0.31	0.12	3.42

Table 3 : Results of Frictional angle obtained from Direct Shear Test

Sample	Frictional angle Φ(deg)	
	Dry	Wet
100% local sand	33.07	36.87
100% kushtia sand	34.44	39.34
100% sylhet sand	33.69	42.14
75% kushtia sand & 25% local sand	33.69	33.48
50% kushtia sand & 50% local sand	32.62	34.22
25% kushtia sand & 75% local sand	33.02	36.87
25% local sand & 75% sylhet sand	30.96	38.66
50% local sand & 50% sylhet sand	32.62	36.87
75% local sand & 25% sylhet sand	34.64	37.1
75% kushtia sand & 25% sylhet sand	32.15	36.87
50% kushtia sand & 50% sylhet sand	30.19	37.78
25% kushtia & 75% sylhet sand	29.25	38.045

3.2 Figures and Graphs

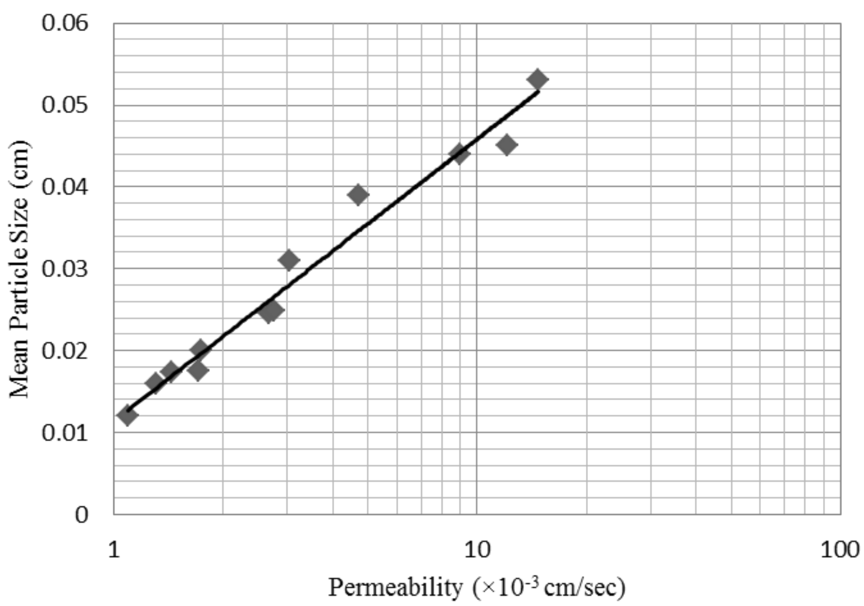

Figure 1: Variation of Permeability with the Mean Particle Size

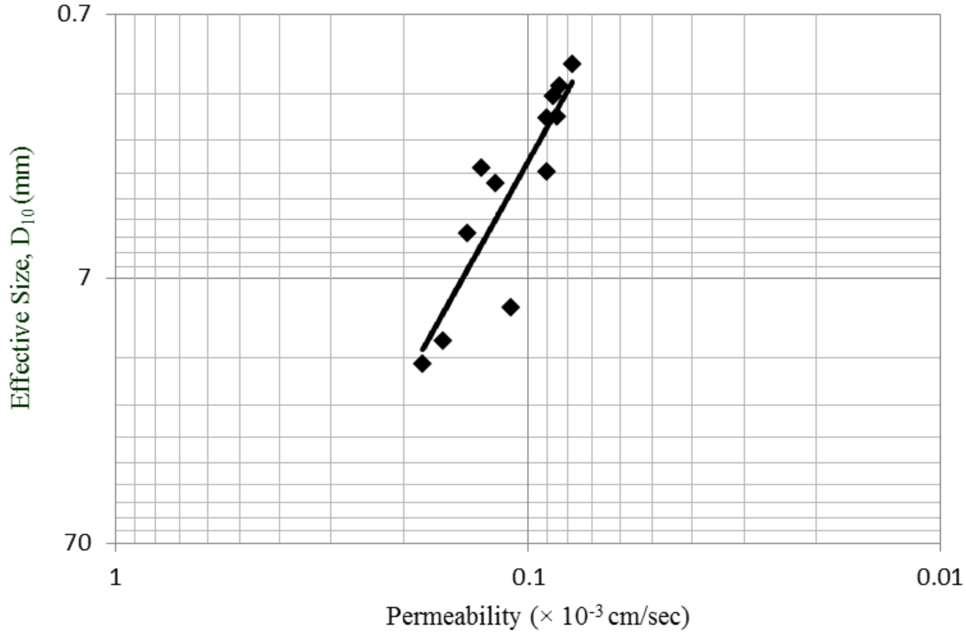

Figure 2: Variation of Permeability with the Effective size

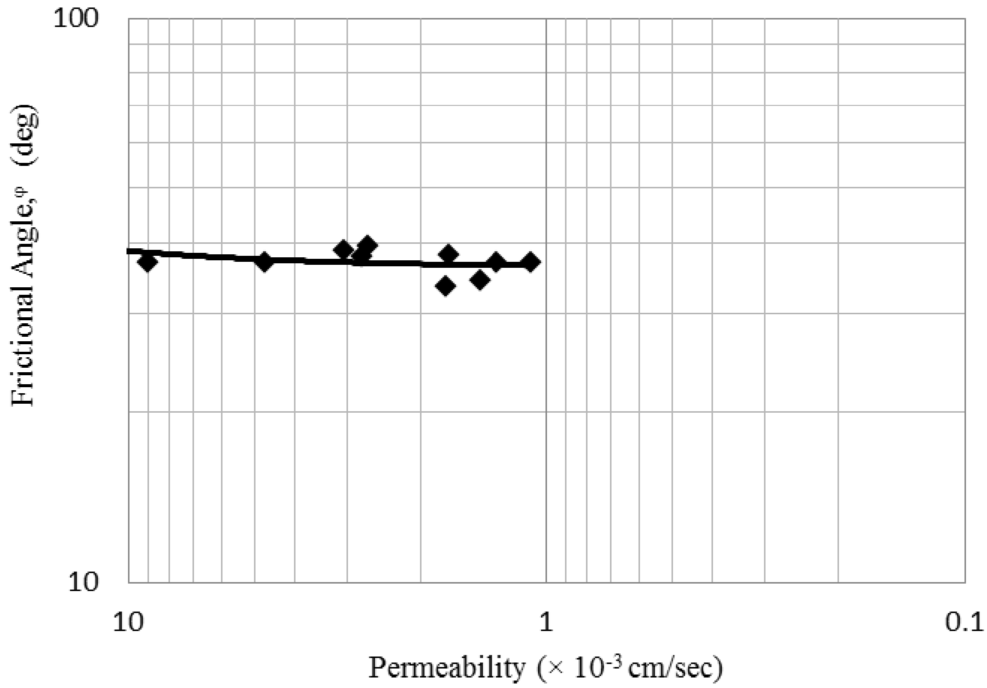

Figure 3: Variation of Frictional angle with Permeability

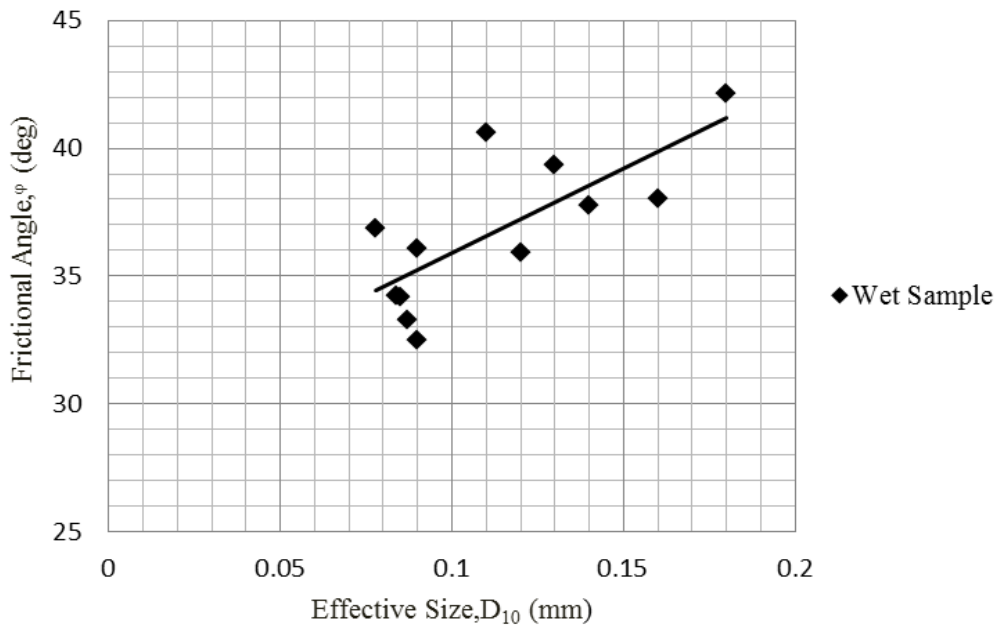

Figure 4: Variation of Frictional angle with Effective Size

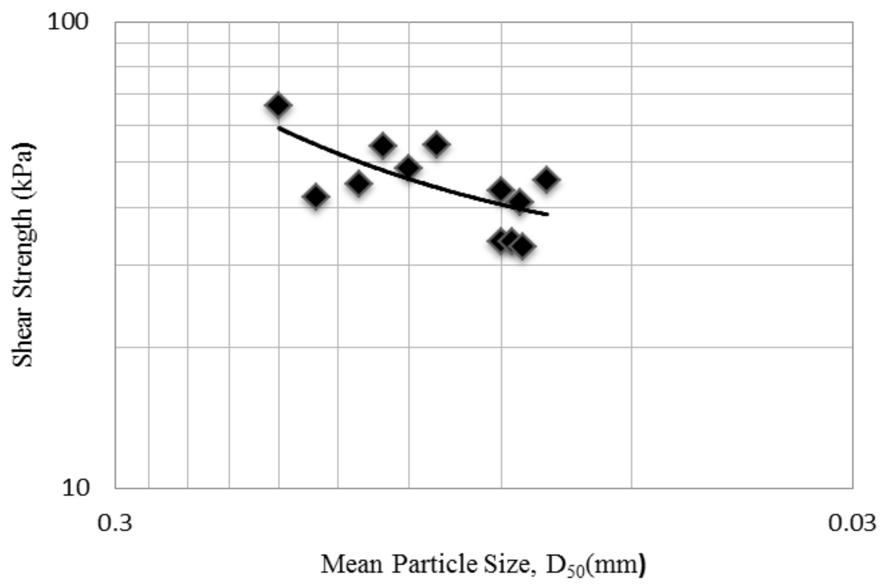

Figure 5: Variation of Shear Strength with the Mean Particle Size

From the results it is shown that the coefficient of permeability varied from 1×10^{-3} cm/sec to 15×10^{-3} cm/sec. Some equations were expressed by the author related to permeability which are shown in Table 4.5

Table 4: Relation between (1) K vs. D_{50} (2) K vs. D_{10} and (3) Su Vs. D_{50}

Sl. No.	Relations	Equations
1	Permeability with Mean Particle Size	$\log(k)= 1.92\log(D_{50}) + 3.56$
2	Permeability with Effective Size	$k =1541(D_{10})^{2.787}$
3	Shear Strength with Mean Particle Size	$S_u=409.61(D_{50})^2+95.404\ D_{50}+28.642$

The permeability equations indicated straight line which holds good for determining coefficient of permeability directly as particle size distribution is easier than permeability measurement rates.

The results showed a direct relation between grain size of particles and frictional angle. It is noted that there is increase in frictional angle with the increase of particle size and decrease in dry density for wet samples.

4. CONCLUSIONS

From the observation of results, following conclusions can be drawn out:

a) The permeability increased significantly with the increase of mean particle size and effective size of sand.

b) Frictional angle increased with the increase of effective size as well as permeability of soil but the variation of frictional angle with permeability of soil is very insignificant.

c) Shear Strength also increased with the increase of mean particle size but the variation is also insignificant in this case.

d) As the changes of value of frictional angle is negligible with the changes of size of sand from fine sand to coarse sand, local sand may be used with certain percentages of kushtia sand in lieu of percentages of sylhet sand in case of sand pile.So it can be suggested that there will be savings a lot of money due to use of local sand with certain percentage of kushtia sand instead of very coarse sand.

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REFERENCES

Ashok Kumar Gupta (2008), “Relationship between the Mean Particle Size, the Size Factor, Optimum Moisture Content, and Permeability of Sandy Soils”, a Conference held on 1-6 october, 2008 in International Association for Computer Methods and Advances in Geomechanics (IACMAG).

Braja M. Das, “ Advanced Soil Mechanics”, McGraw Hill International edition; pp: 404-406

K. Terzaghi, R. B. Peck, G. Mesri, Soil Mechanics In Engineering Practice, Wiley-Interscience, 1996 ; pp: 71-75

Mostefa Kara eta al. (2013),“Contribution of Particle Size Ranges to Sand Friction” ETASR - Engineering, Technolog Applied Science Research Vol. 3, No. 4.

"GEO TEXTILE"-A TREMNDOUS INVENTION OF GEO TECHNICAL ENGINEERING

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ABSTRACT

Filter fabric usually known as Geo textile are a permeable synthetic, textile raw materials .Geo textile are generally used with foundation soil, rock earth or different geotechnical engineering related material. Geo textile plays a significant part in modern technical textile.The use of geotextile in transportation application becomes more popular in modern civil engineering sector.Geotextiles are used in civil engineering earthworks to reinforce of soil, to construct firm bases for temporary and permanent roads and highways, to line ground drains, so that the soil filters itself and prevents soil from filling up the drainpipes and to prevent erosion. The purpose of geotextile is in separation, in drainage, for reinforcement &infiltration.This paper carries the information about the overview of geotextile. For describing the geotextile, the manufacturing process, different application in different sector focusing on civil engineering transportation application, global geotextile market 2012-2016 and the market of geotextile by type, materials, & application are included. The authors have also discussed about the majors players and future of geo market.

Keywords:*Geotextile, Fibers,Fabrics, Properties,Market.*

1. INTRODUCTION

According to the historical record, it is believed that the first applications of geotextiles were woven industrial fabrics used in 1950's. One of the earlist documented cases was a waterfront structure built in florida in 1958. Then, the first nonwoven geotextile was developed in 1968 by the RhonePoulence company in France. It was a comparatively thick needle –Punched polyester, which was used in dam constraction infrance during 1970.***(Hsu-Yeh Huang and Xiao gao,2000***). As we know, the prefix of geotextile , geo means earth and the 'textile' means fabric. Therefore, according to the definitions of ASTM 4439, the geotextile is defined as follows: [***Robert M. Koener, 1998***]

Geotextiles are permeable fabrics which, when used in association with soil, have the ability to separate, filter, reinforce, protect, or drain. Typically made from polypropylene or polyester, geotextile fabrics come in three basic forms: woven (looks like mail bag sacking), needle punched (looks like felt), or heat bonded (looks like ironed felt). Geotextile composites have been introduced and products such as geogrids and meshes have been developed. Overall, these materials are referred to as geosynthetics and each configuration—-geonets, geogrids and others—-can yield benefits in geotechnical and environmental engineering design.

The ASAE (Society for Engineering in Agricultural, Food, and Biological Systems) defines a geotextile as a "fabric or synthetic material placed between the soil and a pipe, gabion, or retaining wall: to enhance water movement and retard soil movement, and as a blanket to add reinforcement and separation." A geotextile should consist of a stable network that retains its relative structure during handling, placement, and long-term service. Other terms that are used by the industry for similar materials and applications are geotextile cloth, agricultural fabric, and geosynthetic.***(http://www.drexel.edu/gri/gmat.htm*)**

2. RAW MATERIALS

Different fribers from both natural as well as synthetic category can be used as geotextiles for various applications.Geotextiles are usually made from one of the four synthetic polymers: polyamide, polyester, polyethylene, and polypropylene or natural materials.*Polyamide* is also known as Nylon 6 and Nylon 6.6.*Polyester* areadditives used in the production of polyester are a) catalysts which increase the speed of polymerization b) phosphatic compounds which reduce thermaldegradation during processing in the molten stage; and c) ageing inhibitors (including carbon black) which increase the U.V. resistance. *Polyethylene*hastwo main groups of polyethylene can be identified: - i) Low density polyethylene (density 920-930 kg/m^3) ii) High density polyethylene (density 940-960 kg/m^3). The polymerization of propylene monomers in the presence of specific catalyst produces the crystalline thermoplastic *polypropylene. Natural fibers* in the form of paper strips, jute nets, wood shavings or wool mulch are being used as geotextiles.*Ramie*isthe fiberswhich have silky luster and have white appearance even in the unbleached condition. They constitute of pure cellulose and possess highest tenacity among all plant fribers.*Jute*is a versatile vegetable fiber which is biodegradable and has the ability to mix with the soil and serve as a nutrient for vegetation. However, their life span can be extended even up to 20 years through different treatments and blending. Thus, it is possible to manufacture designed biodegradable jute geotextile, having specific tenacity, porosity, permeability, transmissibility according to need and location specificity. (***John N.W. 1987)***

3. MANUFACTURING PROCESS OF GEOTEXTILE

Geotextiles are a permeable synthetic material made of textile materials. They are usually made from polymers such as polyester or polypropylene. The geotextiles are further prepared in three different categories – woven fabrics, non-woven fabrics and knitted fabrics.

3.1 Woven Fabrics:

As their name implies, they are manufactured by adopting techniques which are similar to weaving usual clothing textiles. This type has the characteristic appearance of two sets of parallel threads or yarns --.the yarn running along the length is called warp and the one perpendicular is called weft.The majority of low to medium strength woven geo synthetics are manufactured from polypropylene which can be in the form of extruded tape, silt film, monofilament or multifilament. Often a combination of yarn types is used in the warp and weft directions to optimize the performance/cost. Higher permeability is obtained with monofilament and multifilament than with flat construction only. (***wgbhboston , 1996-12***)

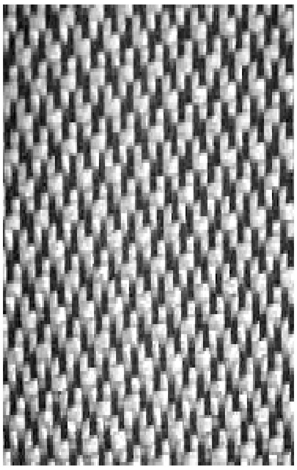
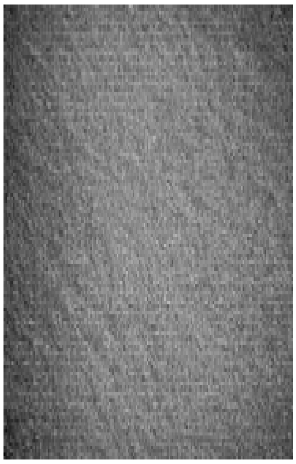

Figure 1: Woven Geotextile   Figure 2: Non-woven Geotextile (***wgbhboston , 1996-12***)

3.2 Nonwoven:

Nonwoven geo-synthetics can be manufactured from either short staple fibre or continuous filament yarn. The fibers can be bonded together by adopting thermal, chemical or mechanical techniques or a combination of techniques. The type of fibre (staple or continuous) used has very little effect on the properties of the non – woven geo synthetics.***(<www@ gmanow.com/pages/desconcept.asp***>) Non-woven geotextiles are manufactured through a process of mechanical interlocking or chemical or thermal bonding of fibres/filaments. Thermally bonded non-wovens contain wide range of opening sizes and a typical thickness of about 0.5-1 mm while chemically bonded non-wovens are comparatively thick usually in the order of 3 mm. On the other hand mechanically bonded non-wovens have a typical thickness in the range of 2-5 mm and also tend to be comparatively heavy because a large quantity of polymer filament is required to provide sufficient number of entangled filament cross wires for adequate bonding.**(<*www@ gmanow.com/pages/desconcept.asp*>)**

3.3 Knitted Fabrics:

Knitted geosynthetics are manufactured using another process which is adopted from the clothing textiles industry, namely that of knitting. In this process interlocking a series of loops of yarn together is made. An example of a knitted fabric is illustrated in figure. Only a very few knitted types are produced. All of the knitted geosynthetics are formed by using the knitting technique in conjunction with some other method of geosynthetics manufacture, such as weaving.

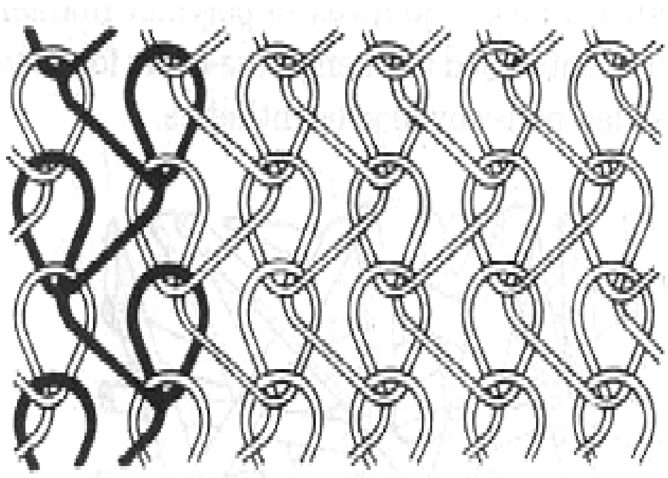

Figure 3:Knitted Geotextile (***Anand S C, August 2000, 53***)

Apart from these three main types of geotextiles, other geosynthetics used are geonets, geogrids, geo-cells, geo membranes, geo composites, etc. each having its own distinct features and used for special applications.(***Anand S C, August 2000, 53***)

4. THE BASIC FUNCTION OF GEOTEXTILE

Geotextiles form one of the two largest groups of geosynthetics. Their rise in growth during the past fifteen years has been nothing short of awesome. They are indeed textiles in the traditional sense, but consist of synthetic fibers rather than natural ones such as cotton, wool, or silk. Thus biodegradation is not a problem. These synthetic fibers are made into a flexible, porous fabric by standard weaving machinery or are matted together in a random, or nonwoven, manner. Some are also knit. The major point is that they are porous to water flow across their manufactured plane and also within their plane, but to a widely varying degree. There are at least 80 specific applications area for geotextiles that have been developed; however, the fabric always performs at least one of five discrete functions: (***robertholtz d., 1997***)

4.1 Separation (TANFEL Report, 1990)

Geotextiles function to prevent mutual mixing between 2 layers of soil having different particle sizes or different properties. Table 2 shows the required properties for separation:

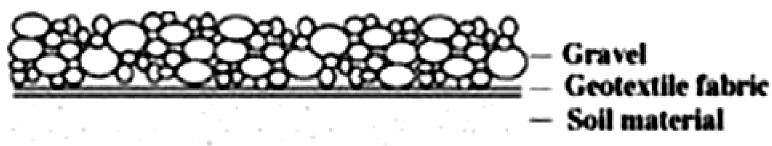

Figure 4:Illustration of a geotextile fabric separating a gravel layer from the underlying soil material. (***~Ohio line/aex-fact/0304.html***)

Table 1: The required properties for separation (**TANFEL Report, 1990**)

• **In different time**	• **Mechanical**	• **Hydraulic**	• **Long-term Performance**
• During installation	• Impact resistance • Elongation at break	• Apparent opening size (A.O.S.) • Thickness	• UV resistance
• During construction	• Puncture resistance • Elongation at	• Apparent opening size	• Chemical stability

	break	(A.O.S.) • Thickness	• UV resistance
• After completion of construction	• Puncture resistance • Tear propagation resistance • Elongation at break	• Apparent opening size (A.O.S.) • Thickness	• Chemical stability • Resistance to decay

4.2 Drainage.

The function of drainage is to gather water, which is not required functionally by the structure, such as rainwater or surplus water in the soil, and discharge it. (***TANFEL Report, 1990***)

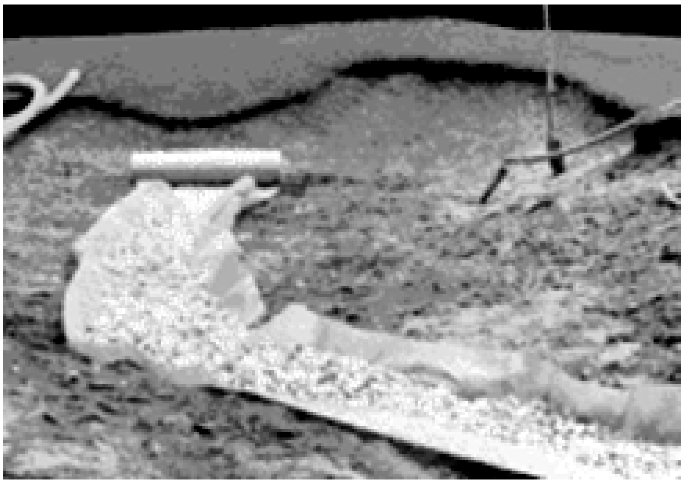

Figure 5: Drainage (***typar.html***)

Table 2:The required properties for drainage (**TANFEL Report, 1990**)

• **Indifferent Function**	• **Mechanical**	• **Hydraulic**	• **Long-term Performance**
• Permanent drainage function	• Influence of normal overburden pressure	• Permeability • Thickness • Apparent opening • size (A.O.S.)	• Chemical properties of water and soil • Chemical stability • Decay resistance
• Temporary drainage function	• Influence of normal overburden pressure	• Permeability • Thickness • Apparent opening • size (A.O.S.)	•

4.3 Filtration :

Filtration involves the establishment of a stable interface between the drain and the surrounding soil. In all soils water flow will induce the movement of fine particles. Initially a portion of this fraction will be halted at the filter interface; some will be halted within the filter itself while the rest will pass into the drain. The geotextile provides an ideal interface for the creation of a reverse filter in the soil adjacent to the geotextile. The complex needle-punched structure of the geotextile provides for the retention of fine particles without reducing the permeability requirement of the drain. (***www.geofabrics.com.au/bidim.html≠filtration***)

Figure 5: Filtration (***www.geofabrics.com.au***)

Table 3: The required properties for Filtration (**TANFEL Report, 1990**)

• **Function**	• **Mechanical filter stability**	• **Hydraulic filter stability**	• **Long-term performance**
• Permanent filter function	• A.O.S. • Thickness	• Geotextile permeability	• Chemical properties of water and soil • Chemical stability • Decay resistance
• Temporary filter function	• A.O.S. • Thickness	• Geotextile permeability	•

4.4 Reinforcement

Due to their high soil fabric friction coefficient and high tensile strength, heavy grades of geotextiles are used to reinforce earth structures allowing the use of local fill material. (***www.cofra.com)***

Figure6: Reinforcement (***www.cofra.com***)

Table 4: The required properties for reinforcement (**TANFEL Report, 1990**)

• **When**	• **Mechanical**	• **Hydraulic**	• **Long-term performance**
• Base failure	• Shear strength of bonding system	• Hydraulic boundary conditions	• Chemical and decay resistance
• Top failure	• Tensile strength of geotextile • Geotextile/ soil friction	• Hydraulic boundary conditions	• Chemical and decay resistance
• Slope failure	• Tensile strength of geotextile • Geotextile/ soil friction	•	• Creep of the geotextile/ soil system • Chemical and decay resistance

4.5 Protection:

Erosion of earth embankments by wave action, currents and repeated drawdown is a constant problem requiring the use of non-erodable protection in the form of rock beaching or mattress structures. Beneath these is placed a layer of geotextile to prevent leaching of fine material. The geotextile is easily placed, even under water. (***http://www.geofabrics.com.au/bidim.html≠embankment***)

Figure 7:Protection (/***bidim.html≠embankment***)

Table 5: The required properties for protection (**TANFEL Report, 1990**)

• **Different section**	• **Mechanical**	• **Long-term performance**

• Tunnel construction	• Burst pressure resistance • Puncture resistance • Abrasion resistance	• Chemically stable: pH=2-13 • Decay resistance
• Landfill and reservoir geomembrane construction	• Puncture resistance • Burst pressure resistance • Friction coefficient	• Chemically stable: pH=2-13 • Decay resistance
• Flat roof construction	• Puncture resistance	• Chemical compatibility

5. APPLICATION OF GEOTEXTILE(GREGORY RICHARDSON N., BARRY CHRISTOPHER R, 1999)

Civil engineering works where geotextiles are employed can be classified into the following categories –

5.1 Road Works:

The basic principles of incorporating geotextiles into a soil mass are the same as those utilized in the design of reinforced concrete by incorporating steel bars. It allow rapid dewatering of the roadbed, the geotextiles need to preserve its permeability without losing its separating functions.

5.2 Railway Works:

The woven fabrics or non-wovens are used to separate the soil from the sub-soil without impeding the ground water circulation where ground is unstable.Enveloping individual layers with fabric prevents the material wandering off sideways due to shocks and vibrations from running trains.

Figure 8: Illustration with geotextile (/stabprod.html)Figure 9: Illustration without geotextile(*/stabprod.html)*

5.3 River Canals and Coastal Works:

Geotextiles protect river banks from erosion due to currents or lapping. When used in conjunction with natural or artificial enrockments, they act as a filter. For erosion prevention, geotextile used can be either woven or nonwoven.

5.4 Drainage:

In civil engineering, the use of geotextiles to filter the soil and a more or less single size granular material to transport water is increasingly seen as a technically and commercially viable alternative to the conventional systems. Geotextiles perform the filter mechanism for drainages in earth dams, in roads and highways, in reservoirs, behind retaining walls, deep drainage trenches and agriculture.

5.5 Sports Field Construction:

Caselon playing fields are synthetic grass surfaces constructed of light resistance polypropylene material with porous or nonporous carboxylated latex backing pile as high as 2.0 to 2.5 cm. Astro Turf is a synthetic turf sport surface made of nylon 6,6 pile fibre knitted into a backing of polyester yarn which provides high strength and dimensional stability. It is claimed that the surface can be used for 10 hr/day for about 10 years or more.

5.6 Agriculture:

It is used for mud control. For the improvement of muddy paths and trails those used by cattle or light traffic, nonwoven fabrics are used and are folded by overlapping to include the pipe or a mass of grit.

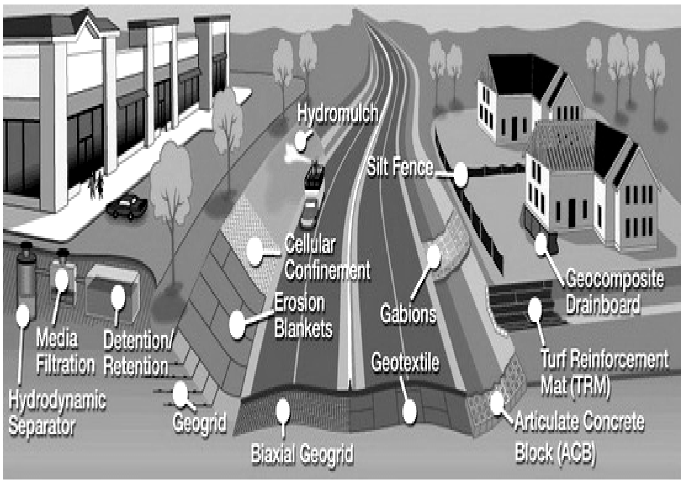

Figure 3: Application of Geotextiles in Civil Engineering [**blogspot.com**]

6. GLOBAL GEOTEXTILE MARKET 2012-2016

TechNavio's analysts forecast the Global Geotextiles market to grow at a CAGR of 10.96 percent over the period 2012-2016. One of the key factors contributing to this market growth is the increasing demand for geotextiles in road and railway construction. The Global Geotextiles market has also been witnessing an increased use of natural geotextiles. However, the rise in raw material prices could pose a challenge to the growth of this market. TechNavio's report, the Global Geotextiles Market 2012-2016, has been prepared based on an in-depth market analysis with inputs from industry experts. The report covers the market in Asia, North America, Europe, and the ROW; it also covers the Global Geotextiles market landscape and its growth prospects in the coming years. The report also includes a discussion of the key vendors operating in this market. The key vendors dominating this space include GSE Holding Inc., OfficineMaccaferriS.p.A., NAUE GmbH& Co. KG., Royal TenCate N.V. The other vendors mentioned in the report are Fibertex Nonwovens A/S, FiberwebPlc, Global Synthetics Pty. Ltd., Low & Bonar plc, Propex Operating Company LLC, Tenax Group Inc.***(Reportbuyer.com, 2012-2016)***

7. GEOTEXTILE MARKET BY TYPES, MATERIALS & APPLICATION

Geotextiles market is a major category within the global geosynthetics industry. It is the most dominating market within geosynthetics with a healthy market growth. The geotextiles market report gives a holistic view about the geotextiles industry while covering diverse market segments which include product types, applications, and materials. There has been a drastic usage of geotextiles in civil engineering applications in different parts of the globe; especially in Asia (China). Nonwoven geotextile is the major market segment for global geotextiles industry followed by woven geotextiles. These two types of products cover a major portion of the global geotextiles market with most of the market players involved in manufacturing and supplying of these product types. Some of the major manufacturers developing these product types include TenCate, NAUE, Huesker Synthetic, Fiberweb, and Propex. Knitted geotextiles form a small part of the global geotextiles industry. Polypropylene, polyester, and polyethylene are the major categories within the geotextiles materials market which are used in the production of geotextile products. Over the past few years, polypropylene and polyester have been the favorite materials for geotextile product manufacturers. Road industry is the dominating applications segment within geotextiles market. More than 50% of the market demand is expected to come from this segment. It is estimated that over the next five years, road industry will govern the geotextiles market share as far as applications are concerned. Some other major applications within geotextiles market include erosion control, waste containment, and pavement repair. The geotextiles market report gives a competitive scenario between the major players by evaluating the objectives of their different developments namely Mergers and acquisitions, Partnerships and agreements, New products launch, and Expansions. This gives a holistic view about the long term plans of the key market players in geotextile industry. Furthermore, the report also profile these companies in order to provide elaborative information such as company overview, financials, products and services, developments and strategies these companies are adopting in order to mark their presence in the market.***(www.researchandmarkets.com)***

8. MAYJOR PLAYERS

Agru America Inc., Amcol International Corporation, Belton Industries Inc., Carthage MillsContech, Engineered Solutions Llc, E. I. Dupont De Nemours And Company, Fibertex Nonwovens A/S, FiberwebPlc, Gundle/Slt Environmental Inc., Hov Environment Solutions Private Limited, Huesker Synthetic Gmbh, Kaytech Engineered Fabrics, Leggett & Platt Inc., Low & Bonar Plc, NaueGmbh& Co. Kg, Nilex, Propex, Strata Systems Inc., Tenax Group, Royal TencateNv, The Dow Chemical Company, Thrace Plastics Co. Sa (www.researchandmarkets.com)

9. FUTURE OF GEOTEXTILE

When looking to future generations of geotextiles, an examination of the role of nanotechnology in the functional enhancement of geotextiles is in order. By reducing fiber diameter down to the nanoscale, an enormous increase in specific surface area to the level of 1000 m2/g is possible. This reduction in dimension and increase in surface areagreatly affects the chemical/biological reactivity and electroactivity of polymeric fibers. Because of the extreme fineness of the fibers, there is an overall impact on the geometric and thus the performance properties of thefabric. There is an explosive growth in worldwide research efforts recognizing the potential nanoeffect that will be created when fibers are reduced to nanoscale. (***Rue de stassart 36, b-1050, brussels***)

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10. CONCLUSION

Textiles are not only clothing the human body but also our mother land in order to protect her. Geo textile and related products have many applications and currently support many civil Engineering applications including roads, airfields, railroads etc. (***Geo textile, From Wikipedia, the free encyclopedia***). On the other hand, Geotextiles are effective tools in the hands of the textile engineer that have proved to solve a myriad of geotechnical problems. (***Kahlid a Meccai&Eayad Al Hasan, 2004***).Most of the Textile Engineer do not know briefly about geo technical Engineering. Not only had the civil Engineer had to know about the geotextile briefly but also Textile Engineer. The author tries to develop a clear idea about geo technical engineering.Extensive awareness should be created among the people about the application of geotextiles.. To explore the potential of geotextile more researches are needed in this field.

11. REFERENCES

Aashto (1996) standard specifications for geotextiles - m288-96 standard specifications for transportation materials and methods of sampling and testing, american association of state transportation and highway officials, washingtond.c., pp 689-692.

Anand S C, (2000), Developments in technical fabrics-Part 2, Knittingint, August 53.

John n.w. m “geo textile”, 1987, pp 12

Tanfel Advertisement Report, 1990, pp 07-15

European committee for standardization (cen), “*geotextiles andgeotextile related products, central secretariat: rue de stassart” 36, b-1050, brussels*.

Gregory richardson n., barrychristopher r. (1998), geotextiles in transportation applications, featured short course, pp 23

Geotextile, From Wikipedia, the free encyclopedia

Kahlid a Meccai&Eayad Al Hasan, (2004), Geo Textile in Transformation Application, Paper presented at the Second Gulf Conference On Roads, Abu Dhabi, March 2004, pp 12

Maxim technologies inc, nonwoven paving fabrics study –final r e p o r t ,december 1997. (<www@ gmanow.com/pages/desconcept.asp>)
Pritched M, Sarsby R W &Anand S C (2000) Textiles in civil engineering: Part-2-Natural fiber geotextile, HANDBOOK OF TECHNICAL TEXTILES, edited by A R Horrocks and S C Anand (Woodhead Publication Ltd, Cambridge, UK), pp388.
Pritchard m, anand s c &sasby r w (1998) novel vegetable fiber geotextile structures for soil reinforcement, procedding conference on textile engineered for performance, umist, april , pp 20-22
Perrier, H.(1986). Use of Soil-Filled Synthetic Pillows for Erosion Protection. Third International Conference on Geotextiles, Vienna, Austria, pp 1115-1119.
Robert holtz d., barrychristopher r., ryan berg r., geosynthetic engineering, bitech publishers ltd. canada, 1997
Robert M. Koener (1998), "Designing with geosynthetics", pp32
Smitht.e., brandont.l., al-qadii.l., lacinab.a., bhuttas.a., hoffmans.e., 1995, *"laboratory behavior of geogrid and geotextile reinforcedflexible pavements; final report," virginia tech, blacksburg, va*.
Hsu-Yeh Huang and Xiao gao, 2000, pp 01-08
Turner, L.W. (1997), Listing of Geotextile Fabric Sources. Biosystems and Agricultural Engineering Web Site: <http:// www.bae.uky.edu/> under "Departmental Research and Extension Information/Resources."
Turner, L.W. (1996), "Reducing Mud Using Highway-Type Filter Materials," AEU-68, Department of Biosystems and Agricultural Engineering, Cooperative Extension Service, College of Agriculture, University of Kentucky, Lexington.
Webboston (1996-12). "Interview with staceyloizeaux".*nova online*. public broadcasting service. retrieved 2009-04-29. "other preparatory operations involve covering/wrapping the columns first with chain link fences and then with geotextile fabric, which is very puncture resistant and has a very high tensile strength. it allows the concrete to move, but it keeps the concrete from flying. the chain link catches the bigger material and the fabric catches the smaller material from flying up and out."
Woolson, Eric (2003-06-30). "Geosynthetics in Storm water Management". Storm water (Santa Barbara, CA: Forester Media).
http://textilelearner.blogspot.com/2012/11/application-of-geotextiles-in-civil.html#ixzz2mEFtwF4g
http://4.bp.blogspot.com/-vlkrvcrvrc5/ulzvkilokei/aaaaaaaaetg/7y6trwlw8/untitleddf.jpg
Reportbuyer.com just published a new market research report: global geotextiles market 2012-2016
http://www.researchandmarkets.com/research/4k5qk2/geotextiles
http://www.drexel.edu/gri/gmat.htm
http://www.ag.ohio-state.edu/~Ohio line/aex-fact/0304.html
http://www.geofabrics.com.au/bidimhtml≠filtration
http://www.geofabrics.com.au/bidimhtml≠embankment
http:// www.cofra.com/typar.html
http://www.acf-envirs.com/stabprod.html

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PERFORMANCE OF ADMIXTURE SOIL AS A BOTTOM LINER OF LANDFILL

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ABSTRACT

A bottom liner of landfill is intended to be a low permeable barrier, which is laid down under engineered landfill sites. Until it deteriorates, the liner retards migration of leachate, and its toxic constituents, into underlying aquifers or nearby rivers, causing spoil of the local water. To improve the quality of local clayey soil (from KUET campus at a depth of 5-7feet below the ground level) as a bottom liner of landfill an admixture (Bentonite clay) has been mixed with that soil at different proportions. The criteria as clay liner such as (coefficient of permeability) k≤ $1x10^{-7}$cm/sec, (plasticity index) PI>7%, 30% fines of which 15% is clay and (water content) W%> plastic limit has been checked with the mix of bentonite respectively 0%, 7% and 10% (Daniel & Coerner,1995). All criteria were not met when checked without mixing any amount of bentonite with sample specimen. After that, 7% bentonite was mixed with the soil and slight improvement was found in the properties of soil. Finally increasing the amount of bentonite as 10% all the criteria were met. Coefficient of permeability was found as $0.54x10^{-7}$cm/s, plasticity index was 16.80%, percentage of fines was greater than 30% and optimum moisture content was greater than plastic limit.

Keywords: *Landfill, bentonite as admixture ,bottom liner and safe disposal of solid waste.*

1. INTRODUCTION

The solid waste management facility regulations require that a groundwater protection system (commonly referred to as a liner system) be installed at all new or expanding landfills. The purpose of a liner system is to prevent leachate from reaching groundwater by collecting leachate for treatment and disposal. By preventing the movement of leachate into groundwater, the bottom liner serves to protect groundwater and surface water from pollution. A bottom liner of landfill is intended to be a low permeable barrier, which is laid down under engineered landfill sites. Until it deteriorates, the liner retards migration of leachate, and its toxic constituents, into underlying aquifers or nearby rivers, causing spoil of the local water.

For hazardous waste landfills, double liners are required. The double liners include a top liner designed to prevent the migration of hazardous constituents into the liner during the active life and post-closure period and bottom liner consisting of one or more layers of clay having conductivity of no more than $1x10^{-7}$ cm/s.

The low hydraulic conductivity of clay minerals makes them potential materials to use as liner landfill for environmental protection. Soils classified as inorganic clay with high plasticity is considered are the suitable material for landfill (Oweis & Khera 1998). If naturally available clay or clayey soil is not suitable for bottom liner, kaolinite or commercially available high swelling clay (Bentonite) can be mixed with local soils or sand. As Bentonite is naturally occurring, locally available, cost-effective and has the chemical composition of altering physical properties of soils an attempt to use it as an admixture has been carried out.

In this study soil samples were collected from KUET campus from 5 to 7 feet depth. A soil to be used as bottom liner is to fulfill some major criteria such as hydraulic conductivity less than $1x10^{-7}$cm/sec, plasticity index greater than 7%, at least 30% fines of which 15% clay and water content should have to be greater than plastic limit. The soil sample was tested against these criteria and found not suitable for using as bottom liner unless admixture used. For that reason Bentonite was mixed with the soil sample at different proportions and found out the optimum percentage of Bentonite mixture.

2. METHODOLOGY

This research work has been conducted upon simple methodology of determining some basic physical parameters of soil which are considered as criteria of being liner of landfill. First, a soil sampling has been made collecting local soil from KUET campus from 5-7 feet depth below the ground level. Then specific gravity, plasticity index, hydraulic conductivity, percentage of fines and remoulding water content were checked for this soil sample. Then mixing 7% bentonite, criteria were checked again and analysed. Upon the results found by 7% mixing, these criteria were rechecked at an increased amount of mixing proportion of bentonite clay as 10%.Then results were analysed and interpreted, the suitable amount of bentonite when requirements met, clay as admixture, has been considered as the optimum mixing proportion.

2.1 Specific gravity test

The specific gravity of a soil is the ratio of the weight in air of a given volume of soil particles to the weight in air of an equal volume of water at a temperature of 4^0c. The specific gravity of a soil is often used in relating of soil to its volume.

2.2 Atterberg limits test

The liquid limit, plastic limit and shrinkage limit of the soil samples were determined based on Atterberg limits test following the testing standard D4318.

2.3 Particle size analysis by hydrometer method

Hydrometer analysis method is applicable for soil particle smaller than 0.075mm. The percentage of sand, silt and clay size can be obtained from this test.

2.4 Determination of remolded water content

The maximum dry density and optimum moisture content of the soil were determined in the laboratory by standard proctor test following the testing standard ASTM D558.

2.5 Permeability Test

The hydraulic conductivity of clay samples were measured using rigid wall perimeter under falling head condition.

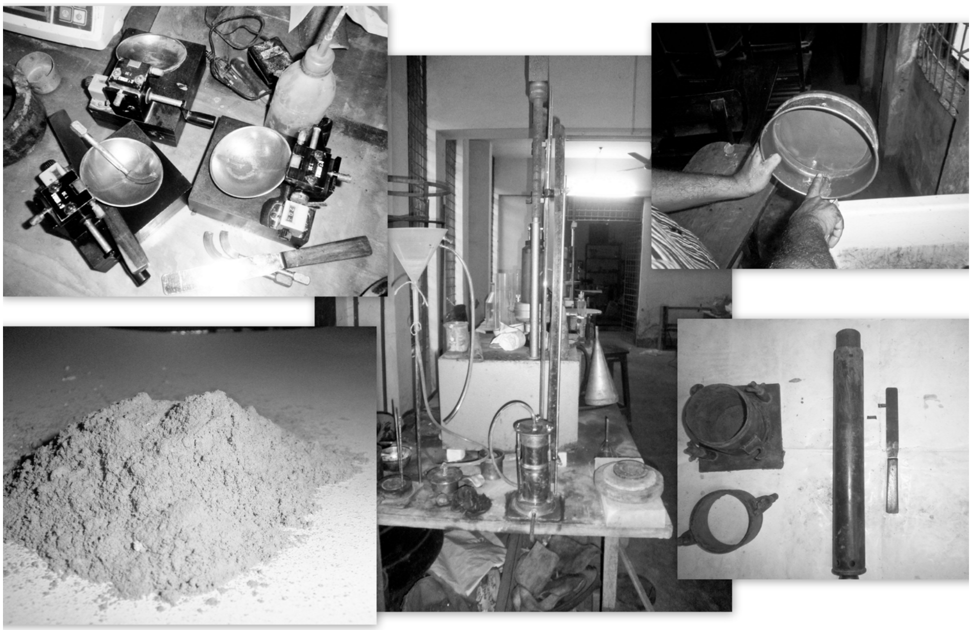

Figure 1: practical photographs of laboratory setup

3. ILLUSTRATIONS

Atterberg limits test

The liquid limit, plastic limit and shrinkage limit of the soil samples were determined based on Atterberg limits test following the testing standard D4318. The liquid limit was found 33.52%. Plasticity index, PI= 6.50%. With the same procedure mixing 7% and 10% bentonite clay with the soil plasticity index were determined 10.99% and 16.80% respectively.

Particle size analysis of bottom liner through hydrometer method

Thus, percentage of fines for 7% and 10% mixing of bentonite clay of that soil were determined (Figure 2).

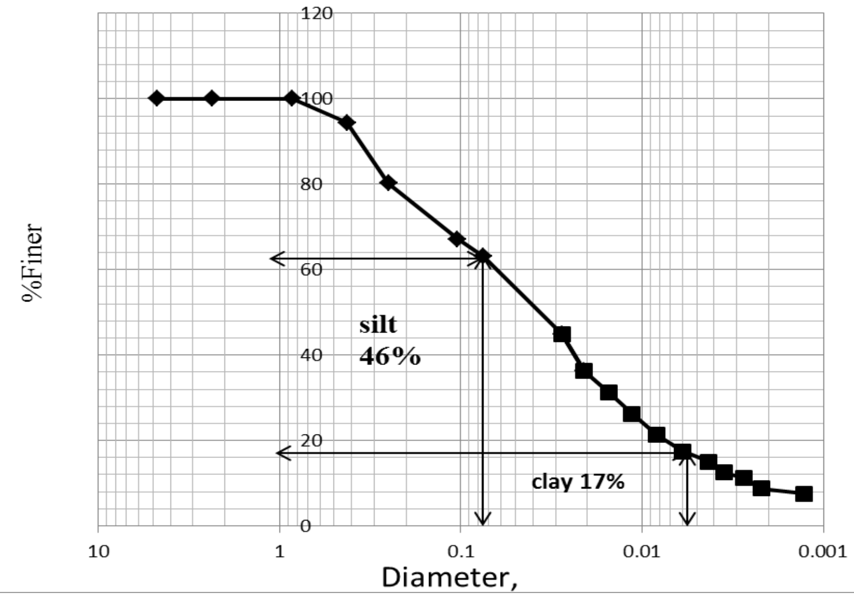

Figure 2: particle size analysis curve by hydrometer method

Determination of compaction properties

The maximum dry density and optimum moisture content of the soil were determined in the laboratory by standard proctor test following the testing standard ASTM D558. Based on the graph (Figure 3), the optimum moisture content was 19% and maximum dry density was 13.30 KN/m^3. Remolding water content of that soil with the mix of 7% and 10% bentonite clay were determined also.

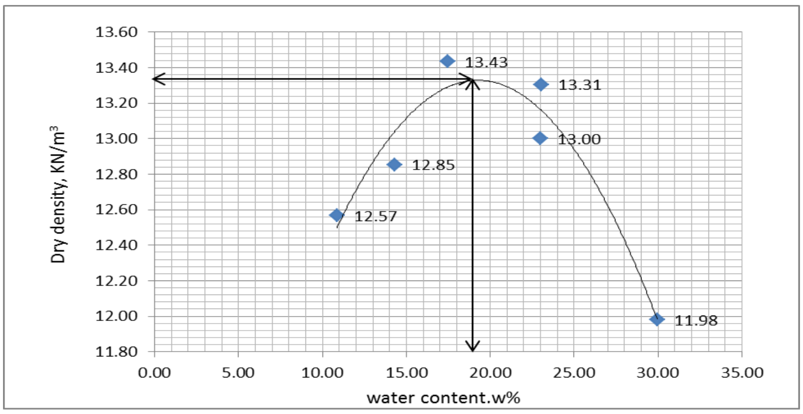

Figure 3: water content vs. dry density relationship

Permeability test

The hydraulic conductivity of clay samples were measured using rigid wall perimeter under falling head condition. The coefficient of permeability of the soil sample were determind for mixing 7% and 10% bentonite clay $2.07 x 10^{-7}$cm/sec and $0.54 x 10^{-7}$cm/sec respectively.

The performance bottom liner depends on the natural characteristics of soil and admixture used. The plasticity index with 0%, 7% and 10% bentonite of soil sample were 6.50%, 10.99% and 16.80% respectively. The plasticity of a soil refers to its capability to behave as a plastic material. And the plasticity index must be greater than 7% (Daniel 1993; Benson et. Al. 1994)

The optimum moisture content and maximum dry density have been obtained by standard compaction test. Standard compaction test keeps the water content at a reasonable value and doesn't increase the value of dry density too much like modified compaction test. The water content and dry density with 0%, 7% and 10% bentonite were found (19%, 13.30KN/m^3) and (18.5%, 15.19 KN/m^3) and(19%, 15.30KN/m^3). At 0% and 7% bentonite mixing the water content was less than plastic limit but by mixing 10% bentonite with soil sample it was found greater than plastic limit.

The percentage of fines was satisfactory which was greater than 30% in every sampling. The percentage of clay was greater than 15% also.

The most important and vital factor of landfill is permeability which denotes the rate of percolation of leachate and other harmful substances. As why, this always should have to be within safe limit. When no admixture was used it was found almost greater than three times of safe limit which was $3.70 x 10^{-7}$cm/s. After using 7% bentonite slight improvement was observed and that was $2.07 x 10^{-7}$cm/s. Finally increasing the amount of bentonite as 10% the coefficient of permeability was found $0.54 x 10^{-7}$cm/s < $1 x 10^{-7}$cm/s.

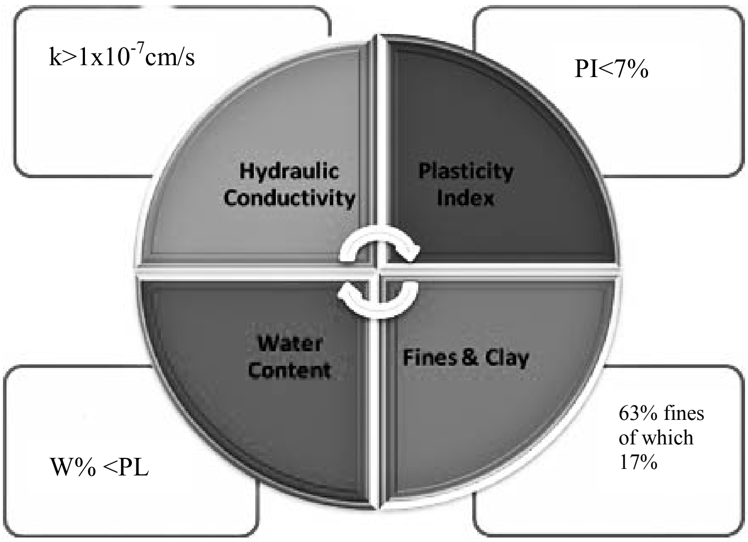

Figure 4: engineering properties of soil as bottom liner when no bentonite used

Above three figures (Figures 4-6) show the change of hydraulic conductivity, plasticity index, % of fines and remolding water content with respect of variation of mixing proportion of bentonite. It was found that when no bentonite was used as admixture all three criteria did not satisfy except percentage of fines. After mixing 7% bentonite only plasticity index had met the condition, additionally with the percentage of fines. Finally, increasing the amount of mixing proportion as 10% of bentonite results were found pretty appreciable. All four criteria were met. So, 10% bentonite can be an optimum proportion while bottom liner designing.

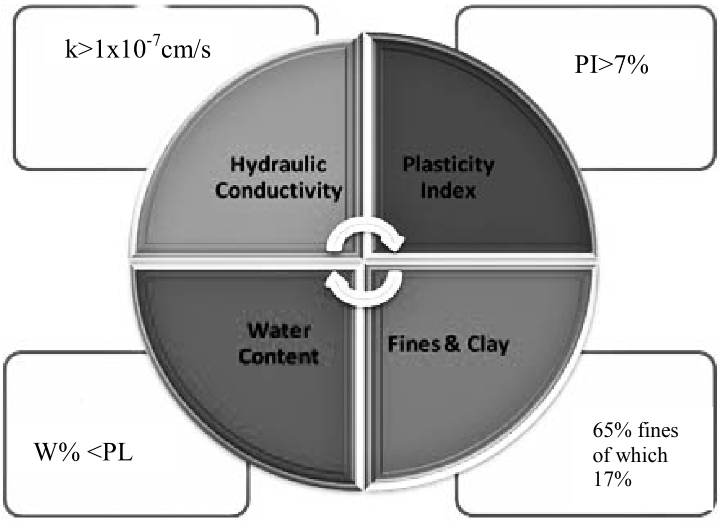

Figure 5: engineering properties of soil as bottom liner when 7% bentonite used

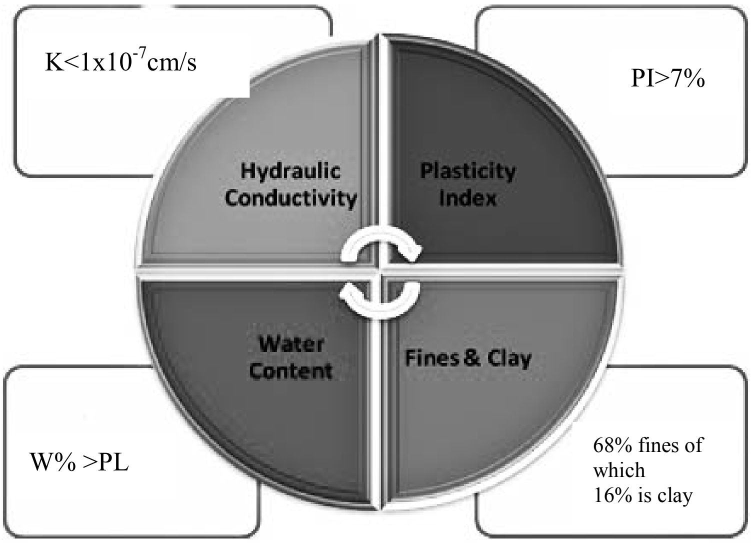

Figure 6: engineering properties of soil as bottom liner when 10% bentonite used

4. CONCLUSIONS

First sampling did not satisfy except percentage of fines. That means the soil sample has fines more than 30% of which at least 15% clay. When second sampling was made with mixing 7% bentonite plasticity index met the requiring condition but the water content had been less than plastic limit. In third sampling all four requirements were satisfied and hence it can be considered that 10% use of bentonite is the optimum for that kind of soil as a bottom liner.

REFERENCES

Daniel, D.E and Koerner, R.M. (1995). Waste Containment Facilities, Guidance for Construction Quality Assurance and Quality Control of Liner and Cover System ASCE Press, New York, N.Y, pp.13, 25-27.

Oweis, I.S., & Khera, R.P., (1998), Geotechnology of Waste Management, 2nd Edition, PWS Publishing Company, USA.

A COMPARATIVE STUDY ON THE LOAD CARRYING CAPACITY OF RC BORED PILES

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ABSTRACT

The need for deep foundation including pile foundation is a very important structural option particularly for weak/soft soil underlying the existing ground surface. Piles are structural members made of reinforced concrete, steel or timber and capable of carrying vertical as well as lateral loads. Through piles, the foundation load can be transferred to harder and reliable soil strata that ensure the effective transfer of super structural load to the ground.

Generally soil is a complex materials and its characteristics varies with its parameters such as cohesion (c), angle of friction (ϕ), number of layers in which the pile is installed. The pile load capacity depends on several factors including soil type and texture, state of stress, pore pressure, vibration, movement, settlement characteristics etc. and ultimately on friction between soil and pile and point bearing capacity. The relevant soil parameters may be varied depending on the nature of soil and their effects on load carrying capacity can be evaluated by using different methods:

The study report covers the theoretical evaluation of pile load carrying capacity considering various soil properties. The soil parameter, cohesion (c) & angle of internal friction value (ϕ) has been varied to different extent ($c = 0.5, 1.0, 1.5$ and $\phi = 10^0, 20^0, 30^0, 40^0$) to observe their effects on load capacity of pile. The pile shaft capacity has been evaluated with the use of different method namely α, β & λ methods and the comparison have been made. On the basis of filed as well as laboratory investigation report of soil samples collected from 15 different bore hole location, bearing capacity of piles of different size and length has been calculated by using SPT equation to compare with the values obtained by the general method.

Using various soil parameters, a design aid table for the determination of bearing capacity of pile of various length & size under different soil properties has been presented. It is observed that, for the similar soil condition, skin friction values obtained by α method is observed to be higher (30 to 60%) than that obtained from λ method. A co-relationship has been established between the pile capacities obtained by general method and from field SPT values which may be of use for design of pile foundation.

Keywords: *RC Bored Piles, SPT Values, Skin Friction, Point Bearing Capacity, Cohesion*

1. INTRODUCTION

For The use of piles is man's oldest method of overcoming the difficulties on soft soils. Although it dates back to fore historic lake villages, until the late nineteenth century; the design of pile foundation was based entirely on experience or even divine providence. In recent years, the increasing demand on the foundation engineer to predict reliably the behavior of pile design has stimulated more sophisticated theoretical research on to the interaction between a pile or piles and the embedding soil, so that a large volume empirical knowledge is now balanced by a comparable theoretical understanding.

Piles are structural members made of steel, concrete or timber which are used to support the weight of the structures. Pile foundations are deep and are typically used under high load or weak sacrificial soils. Pile load capacity is the ability to carry vertical load coming from the super structure and lateral loads. It depends on the soil condition at site. Generally soil is a complex materials and its characteristics varies with its parameters such as cohesion (C), angle of friction (ϕ), number of layers in which the pile is installed. The pile load capacity depends on friction between soil and pile and point bearing capacity. Other parameters of pile load capacity are number of blow, falls of hammer, penetration due to hammer blow etc.

The choice of the pile is governed by site conditions, economics and time considerations. For loose to medium sandy and silty starta, bored compaction piles should be used since in such piles, the compaction process increases the load bearing capacity of piles. In case of expansive soils e.g. black cotton soils or filled up soils, under-reamed piles with bulbs provide a good anchorage. It is found that provision of bulbs in the under-reamed piles increases the lateral load capacity of piles. For the multistoried apartments, high chimneys or heavier structures, multi under-reamed piles are generally used. In general, under-reamed piles can be used where the structures are subjected to various loading conditions including those due to wind and seismic forces. These piles can be constructed as a depending upon the actual requirements.

An extensive field test should be carried out before taking up the pile construction on site. Sufficient bore holes with required depths should be made on site to study the soil properties. The extensive laboratory tests should be carried out on the bored soil samples to study the permeability, shear and cohesive properties of soil. The extent of soil exploration depends on the type of structure, possible loads, layout of structure, economics etc. Different types of piles are used in various construction areas depending on ground disturbance, material of constructions and construction practice. The bearing capacity of piles can be determined by using static, dynamic formula and also by using SPT values of the subsoil. The theoretical load carrying capacity of piles can be verified and confirmed by pile load test results.

The main objective of the study is to determine the load carrying capacity of bored pile generally used in foundation works. The specific objectives of the study are as follows:

- To determine the load carrying capacity of pile considering static formula.
- To evaluate the load carrying capacity of pile from soil SPT values.
- Comparison of the different methodologies used for pile capacity evaluation.
- To establish the correlation of the load carrying capacity of the piles using different methods.

2. STATEMENT OF THE PROBLEM

The study covers the theoretical approach to know the different existing methodologies / formulae for determining the bearing capacity of pile under different subsoil condition of soil. The load capacity of pile depends on several factors including soil type and texture, state of stress, pore pressure, vibration, movement / settlement characteristics etc .The relevant soil parameters can be varied depending on the nature of soil and their efforts on load carrying capacity can be evaluated by using different methods .On the basis of the pile load carrying capacity results, a correlation among the existing methodologies can be established.

3. STANDARD PENETRATION TEST

The standard penetration test is the most commonly used in-situ test, especially for cohesion less soils which cannot be easily sampled. The test is extremely useful for determining the relative density and the angle of shearing resistance of cohesion less soils. It can also be used to determine the unconfined compressive strength of cohesive soils. The standard penetration test is conducted in a bore hole using a split- spoon. The split-spoon sampler is connected to string of drill rods and lowered into the bottom of the bore hole which is drilled and cleaned in advance. When the bore hole has been drilled to the desired depth, the drilling tools are removed and the sampler is lowered to the bottom of the hole. The sampler is driven into the soil by a drop hammer of 63.5 kg mass falling through a height of 750 mm at the rate of 30 blows per minute. The number of hammer blows required to drive 150 mm of the sample is counted. The sampler is further driven by 150 mm and the number of blows recorded. Likewise, the sampler is once again further driven by 150 mm and the number of blows recorded. The number of blows recorded for the first 150 mm is disregarded. The number of blows recorded for the last two intervals are added to give the standard penetration number (N). In other words, the standard penetration number is equal to the number of blows required for 300 mm of penetration beyond a seating drive of 150 mm. If number of blows for 150 mm drive exceeds 50, it is taken as refusal and the test is discontinued. The observed / field N values may be greater or less than the actual value due to various reasons. Two types of correction i.e. correction for (i) dialatency and (ii) overburden pressure are applied to the observed value.

4. LOAD CARRYING CAPACITY OF PILES

The load transmitted to the soil along the length of the pile is called as the ultimate friction (Q_f) and that transmitted to the base is called point load (Q_b). Total ultimate static load expressed by following expression **(Ref.: Fig. 1):**

$Q_u = Q_b + Q_f = q_b.A_b + f_s.A_s$

Where, qb = *ultimate unit bearing capacity*

Ab = *bearing tip area of the pilr*

As = *total surface area of file*

fs = *unit skin friction*

Q_f = *skin friction of pile* = $\Sigma(\Delta L)\ (a_s.S_s)$

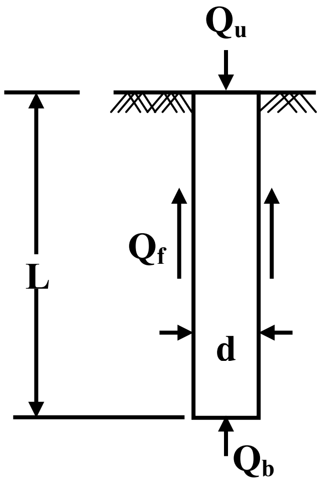

Fig. 1: Reactive Forces on Pile.

The load carrying capacity of pile can be determined by following three ways,

i. By using Static Formula,
ii. By using Dynamic Formula &
iii. By using SPT Values.

4.1 Static Methods

The static bearing capacity of a pile is calculated from the consideration of the properties of the soil medium through the pile passes. Thus the bearing capacity is calculated as the sum of the total ultimate skin friction resistance and the total ultimate end bearing resistance.

Thus, $Q_u = A_p(CN_c + 0.4\gamma BN_\gamma + qN_q) + \Sigma(\Delta L)\ (a_s.S_s)$ (Mittal, S., 1988)

$q_b = (CNc + 0.4\gamma BN\gamma + qN_q)$

A_p = *Area of pile tip*

ΔL = *increment of pile length*

a_s = *perimeter of the pile*

S_s = *unit shaft resistance*

γ = *unit weight of soil*

B = *least dimension of pile tip*

C = *cohesion of soil*

L = *Depth of embedment pile*

Nc, N_q, N_γ = *bearing capacity factors.*

The static pile capacity depends on the soil parameter cohesion and angle of friction.

Skin Friction

The skin resistance part of the above equation is currently computed using both a combination of total and effective or only effective stresses. The following factors on which the skin friction depend on the factors including

i. Type of soil
ii. Depth in the ground
iii. Degree of natural consolidation and saturation.
iv. Shape of pile and amount of compaction by it.
v. Surface texture of pile and
vi. Time interval between driving and testing.

Three methods namely α, λ and β methods are used in the skin resistance capacity determination (Bowles, 1988).

(a) The α-method

The α method was proposed by Tomlison (1971), and basically the skin resistance is computed as

$S_s = \alpha c + \bar{q}\, K \tan\delta$

*α = adhesion factors from **Fig. 2***

C = avg. cohesion (or s_u) for soil structure

$\bar{q}$ *= Effective vertical stress on element ΔL*

K = co-efficient of lateral earth pressure ranging from K_0 to about 1.75.

Where, $K_0 = (1-\sin\varphi')\sqrt{OCR}$

δ = effective friction angle between soil and pile material (use φ' = δ)

(b) The λ-method

Vijayvergia and Focht (1972) presented a method of obtaining the skin friction Ss of a pile in clay as,

$S_s = \lambda(\bar{q} + 2s_u)$

*λ = coefficient which can be obtain from **Fig. 3***

s_u = avg. cohesion of soil or undrained shear strength of soil.

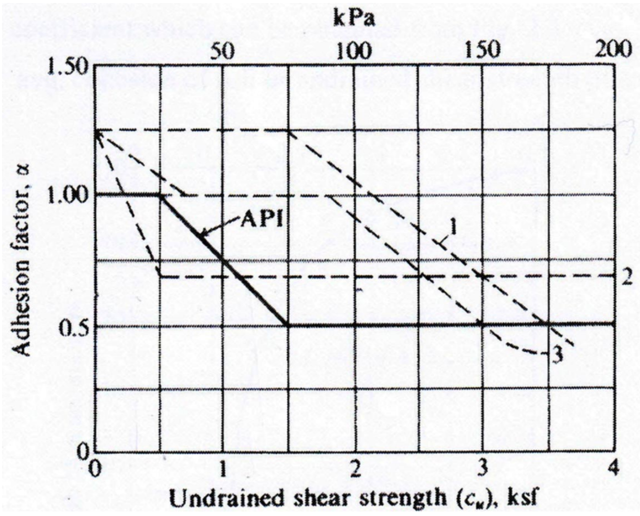

Fig. 2: Relationship between soil and adhesion factor

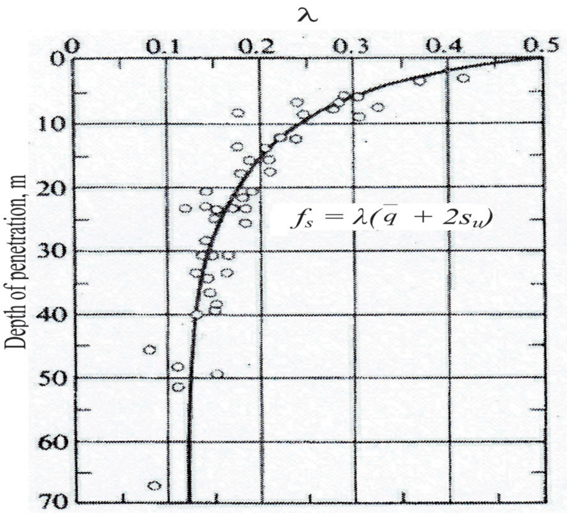

Fig. 3: λ coefficient depending on pile penetration.

(c) The β-method

Burland (1973) developed a simple design equation written as,

$S_s = K\bar{q}\tan\delta$

Where, $K\bar{q} = \beta$, $\beta = 0.18 + 0.0065\, D_r$ or $K = 0.5 + 0.008\, D_r$, D_r = relative density (%) K & δ depends on pile materials.

4.2 Dynamic method

Numerous empirical formulae have been developed as an attempt to predict the capacity of a driven pile from its resistance to penetration. The basis of the most of these formulae is a transfer of kinetic energy of the hammer to the pile and to the soil.

4.3 Methods based on SPT

Meyerhof (1956) suggests the following equations for single piles in granular soils based on SPT values,
For displacement piles

$$Q_u = 400NA_b + 2\overline{N}A_s$$

For H-piles

$$Q_u = 400NA_b + 2\overline{N}A_s$$

For Bored piles

$$Q_u = 133NA_b + 0.67\overline{N}A_s$$

Where,

N = *Average SPT value below pile tip*

$\overline{N}$ = *Average SPT value along the pile shaft*

A_b = *Base area of pile in* m^2

F.S = 4 for driven piles

F. S = 2.5 for bored piles

5. END BEARING CAPACITY BASED ON SPT VALUE

According to Meyerhof (1976), the ultimate end resistance Q_p in tones of driven piles can be estimated by the following relationship:

For Sand, $Q_p = (\frac{0.4\overline{N}}{B}) D_f A_p \leq 4\overline{N}A_p$

For Cohesion less soil,

$$Q_p = (\frac{0.4\overline{N}}{B}) D_f A_p \leq 3\overline{N}A_p$$

Where, $\overline{N} = C_N N$

$$C_N = 0.77\log_{10}(\frac{20}{\sigma'_v})$$

$$\sigma'_v \geq 0.25tsf$$

Friction Capacity on perimeter surface

The friction capacity of a pile can be estimated by using the following relationship:

$$Q_f = f_s \text{ (perimeter) (embedment length)}$$

$$f_s = (\frac{\overline{N}}{50}) \leq 1 \text{ tsf}$$

Where,

$\overline{N}$ = *Avg. corrected standard penetration*

A_p = *pile tip area*

B = *Pile width area*

D_f = *Depth of pile*

σ'_v = *Effective overburden pressure in tsf.*

Determination of Bearing Capacity of Piles Using SPT Values:

In addition to the calculation of Bearing Capacity of pile by using General method, the pile capacity can also be calculated directly from the field SPT values as per following equation:

$$Q_u = 133\,NA_b + 0.67\,\overline{N}A_s \;\ldots\ldots\ldots\; (a)$$

N = *Average SPT value below pile tip*

$\overline{N}$ = *Average SPT value along the pile shaft*

A_b = *Base area of pile in* m^2

A_s = *Shaft surface area in* m^2

A typical bore log chart for a soil exploration is shown in **Fig. 4**

SUB SOIL EXPLORATION

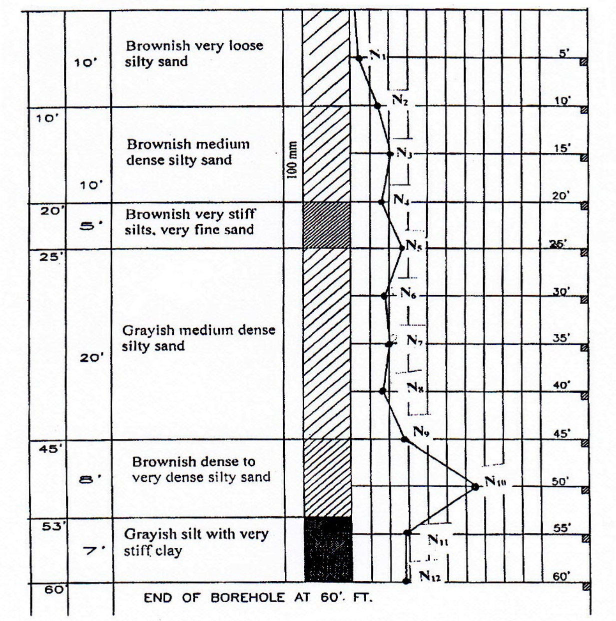

Fig. 4: Typical Bore Log Chart

The step by step procedure for calculating the Bearing Capacity of pile by using field N values as follows:

Step 1: For specific pile length says $L = 55ft$, compute $\overline{N}$ as

$\overline{N} = (N_1 + N_2 + N_3 + N_4 + N_5 + N_6 + N_7 + N_8 + N_9 + N_{11})/10$. Here, N_{10} value is not counted due to its abrupt change of value.

Step 2: Compute N as $(N_{11} + N_{12})/2$

Step 3: Compute pile capacity Q_u by using equation (a)

Step 4: Compute allowable pile capacity $Q_a = Q_u/2.5$.

1. Determination of bearing Capacity

(a) Using General Equation

Out of the different static methods mentioned in literature review list, the following general equation can be used to determine the bearing capacity of the pile.

$$Q_u = Q_p + Q_f$$
$$= A_p(CN_c + 0.4\gamma N_\gamma + qN_q) + \sum(\Delta L).(a_s.f_s)$$
$$f_s = K\bar{q}\tan\delta$$
$$\delta = 3/4\varphi$$
$$N_q = \tan^2(45^0 + \varphi'/2)e^{\pi\tan\varphi'}$$
$$N_c = (N_q - 1)\cot\varphi'$$
$$N_\gamma = 2(N_q + 1)\tan\varphi'$$

The input variables are:

Diameter (ft) = 18″, 20″, 24″ etc.

C (ksf) = 0.5, 1.0, 1.5 etc.

φ (Degree) = 10, 20, 30, 40 etc.

$\gamma' = \gamma - \gamma_w = 0.0625\ k/ft^3$

Length (ft) = 40, 60, 80, 90, 100 etc.

By using the different soil parameters mentioned above, a design aid table can be made with the help of suitable computer program (Excel).

(b) Determination of Pile Shaft Resistance by Two Different Methods

As mentioned earlier, the pile shaft (skin) resistance can be computed by three different methods namely α, β, λ method. α & λ methods are observed to use same soil parameters while β method is different and used only for sandy soil. For the determination of pile shaft resistance α & λ method has been used for comparative study. The input variables were,

Length (ft) = 30, 40, 50, 60, 70, 80, 90, Cohesion *C (ksf) = 0.5, 0.75, 1.25, 1.5, δ (degree) = 15, 20, 30.*

Table 1 shows the pile shaft resistance as per α & λ method for some limited number of variable parameters (L = 50′-90′, K = 0.5 to 1.5 ksf, $\delta = 20^0$ & 30^0)

Table 1: Pile Shaft Resistance by α and λ Method

L(ft)	C(ksf)	γ(k/ft³)	δ(Deg)	OCR	λ	α	K	δ(Rad)	q(ksf)	fs(α) (ksf)	fs(λ) (ksf)	Difference (%)
50	0.5	0.11	20	2	0.2	1	0.93	0.35	5.5	2.36	1.3	44.99
50	0.75	0.11	20	2	0.2	0.875	0.93	0.35	5.5	2.52	1.4	44.43
50	1	0.11	20	2	0.2	0.75	0.93	0.35	5.5	2.61	1.5	42.60
50	1.25	0.11	20	2	0.2	0.625	0.93	0.35	5.5	2.64	1.6	39.50
50	1.5	0.11	20	2	0.2	0.5	0.93	0.35	5.5	2.61	1.7	34.95
70	0.5	0.11	20	2	0.17	1	0.93	0.35	7.7	3.11	1.479	52.42
70	0.75	0.11	20	2	0.17	0.875	0.93	0.35	7.7	3.26	1.564	52.09
70	1	0.11	20	2	0.17	0.75	0.93	0.35	7.7	3.36	1.649	50.90
70	1.25	0.11	20	2	0.17	0.625	0.93	0.35	7.7	3.39	1.734	48.85
70	1.5	0.11	20	2	0.17	0.5	0.93	0.35	7.7	3.36	1.819	45.84
90	0.5	0.11	20	2	0.14	1	0.93	0.35	9.9	3.85	1.526	60.40
90	0.75	0.11	20	2	0.14	0.875	0.93	0.35	9.9	4.01	1.596	60.20
90	1	0.11	20	2	0.14	0.75	0.93	0.35	9.9	4.10	1.666	59.40
90	1.25	0.11	20	2	0.14	0.625	0.93	0.35	9.9	4.14	1.736	58.02
90	1.5	0.11	20	2	0.14	0.5	0.93	0.35	9.9	4.10	1.809	55.99
50	0.5	0.11	30	2	0.2	1	0.71	0.52	5.5	2.75	1.3	52.65
50	0.75	0.11	30	2	0.2	0.875	0.71	0.52	5.5	2.90	1.4	51.76
50	1	0.11	30	2	0.2	0.75	0.71	0.52	5.5	3.00	1.5	49.93
50	1.25	0.11	30	2	0.2	0.625	0.71	0.52	5.5	3.03	1.6	47.14
50	1.5	0.11	30	2	0.2	0.5	0.71	0.52	5.5	3.00	1.7	43.25
70	0.5	0.11	30	2	0.17	1	0.71	0.52	7.7	3.64	1.479	59.41
70	0.75	0.11	30	2	0.17	0.875	0.71	0.52	7.7	3.80	1.564	58.84
70	1	0.11	30	2	0.17	0.75	0.71	0.52	7.7	3.89	1.649	57.65
70	1.25	0.11	30	2	0.17	0.625	0.71	0.52	7.7	3.93	1.734	55.82
70	1.5	0.11	30	2	0.17	0.50	0.71	0.52	7.7	3.89	1.819	53.29
90	0.5	0.11	30	2	0.14	1	0.71	0.52	9.9	4.54	1.526	66.40
90	0.75	0.11	30	2	0.14	0.875	0.71	0.52	9.9	4.70	1.596	66.03
90	1	0.11	30	2	0.14	0.75	0.71	0.52	9.9	4.79	1.666	65.23
90	1.25	0.11	30	2	0.14	0.625	0.71	0.52	9.9	4.82	1.736	64.01
90	1.5	0.11	30	2	0.14	0.5	0.71	0.52	9.9	4.79	1.809	62.31

From the above table, it is seen that for the similar soil properties and conditions, the pile skin resistance calculated by α- method is always seen to be higher than λ-method. The skin resistance obtained by α-method is reported to be 30% to 60% higher than λ-method.

(c) Correlation between the Methods for Pile Load Bearing Capacity

A soil test report generally provide the end bearing & shaft resistance of pile at different depth level and also the ultimate load capacity of pile (Q_u) after conducting different laboratory test of the explored soil samples. The report also contains the SPT graph i.e. the field N values at different depth level. Soil test report from 15 different locations adjacent to Chittagong city has been collected to get the Pile capacity

value (Q_u) as obtained from bored soil sample properties by using general equation. Again from the same location; pile capacity has been calculated from the field SPT values as per relevant equation. For soil samples from 15 different locations and for 83 nos. Q_u*(General)* values, the corresponding 83 nos. Q_u *(SPT)* values has been calculated and inserted in tabular form for different location (**Ref. Appendix A, Table A1 - A15).** A graphical correlation has been established with Q_u*(General)* as *Y* axis and Q_u*(SPT)* as *X* axis **(Ref.: Fig. 5)**. The relation between Q_u*(General)* and Q_u*(SPT)* can be expressed as
$Y = 1.399x + 2.934, R^2 = 0.833$

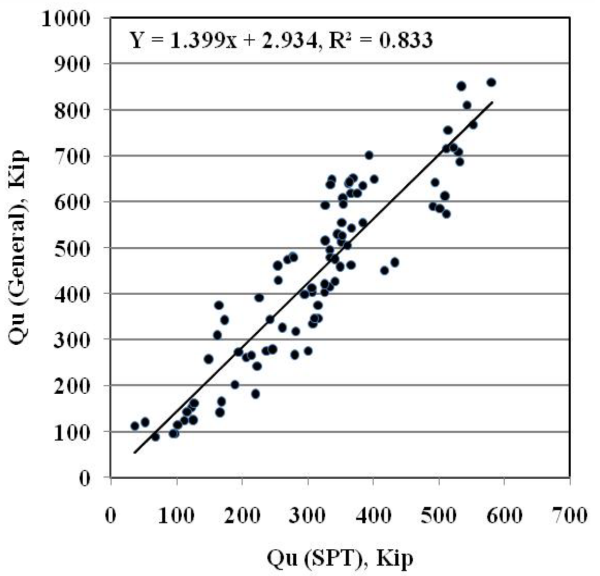

Fig. 5: Relation between Q_u(General)& Q_u(SPT) values

6. SUMMARY AND CONCLUSION

6.1 General

The main objective of the study is to determine the load carrying capacity of piles by using different methods. The pile capacity has been evaluated from the properties of the soil under and around the pile by using static equation & SPT equation. The static equation requires the field test data and laboratory test results of the soil sample collected from the field. On the other hand, SPT method involves with the direct field test data i.e. N value for the evaluation of pile capacity. Soil parameters such as cohesion (c), angle of internal friction (φ) has been varied to different extent to predict their effect on ultimate load carrying capacity of pile.

A total nos. of 83 data for Q_u*(Pile Capacity)* is obtained from soil test report conducted at 15 different locations. Corresponding to the same data, Q_u*(SPT)* from the respective field N value information chart has been calculated. With the use of the collected / computed data, a correlation has been established.

6.2 Conclusion

On the basis of the limited nos. of variable i.e. soil properties / field observation data studied; the following concluding remarks can be made:

(a) A relation between the pile capacity Q_u*(General) (Obtained by general method)* and Q_u*(SPT) (from field SPT value)* may be expressed as, $Y = 1.399x +2.934, R^2 = 0.833$

(b) Q_u*(General)* values are observed to be always higher than Q_u*(SPT)* values. The Q_u*(General)* values are reported as 40% to 80% higher than Q_u*(SPT)* values.

(c) For the similar soil properties and conditions, the pile skin resistance calculated by α- method is always seen to be higher than λ-method. The skin resistance obtained by α- method is reported to be 30% to 60% higher than λ-method.

Reference

Burland, J. B. (1973), "*Shaft Friction piles in clay - A simple Fundamental Approach*", Ground Engineering, Vol. 6, No. 3, pp. 30-42.

Josheph E. Bowles (1988), *Foundation Analysis and Design*. 4th Edition, 1988.

Meyerhof, G. G. (1956), "*Penetration Test and Bearing capacity of cohesion less soil*", JSMFD, ASCE, Vol. 82, SM 1. 1956.

Meyerhof G. G. (1976), *Bearing capacity and Settlement of Pile foundation*, JGED, ASCE, Vol. 102, GT 3, 1976.

Satyendra Mittal (1988), *Pile Foundations Design and Construction*, 1st Edition, 1988.

Tomlinson, M, J. (1971), "*Some effects of pile driving on Skin friction*", "Proc. Conference on behavior of piles", ICE, London, pp. 107-114.

Vijavergiya, V. N, and J. A. Focht, Jr. (1972), "*A new way to predict capacity of piles in clay*", OTC Paper 1718, 4th Offshore Technology Conference, Honston, TX.

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EFFECT OF SAMPLE SIZE ON THE MACRO AND MICRO-SCALE BEHAVIOR OF GRANULAR MATERIALS USING 3D DEM

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ABSTRACT

This paper presents the effect of sample size on the macro- and micro-scale behaviors of granular materials such as sand using the three-dimensional (3D) Discrete Element Method (DEM). Three numerical samples of different sizes were prepared using spheres as particles. During the random sample generation, spheres of different radii were placed randomly in a cube without any contact. After the random generation of three successful samples, they were subjected to isotropic compression by moving their boundaries inward. Simulations of true triaxial compression tests were carried out using these samples to investigate their behaviors both at macro- and micro-scale. The numerical results indicate that the tress-strain-dilative behavior agrees well with the laboratory based experimental results qualitatively for different sample sizes. Small size sample attains the highest strength and it decreases as the sample size increases. It is noted that the evolution of coordination number is the highest for small size sample and it decreases as the sample size increases.

Keywords: *Sample size, micro-scale, granular materials, simulations*

1. INTRODUCTION

Sample size is known to have influence on the mechanical behaviors of granular materials such as sand, both at macro- and micro-scale. The sample size effect of a granular system at macro-scale is often studied using the conventional experimental devices and techniques in the laboratories. For example, Jansma (1988) reported that sample size has effect on the macro behavior and indicated that small size samples gain higher strength than large size samples. Hu et al. (2011) indicated that pre-peak behavior is not affected by the specimen size, whereas post-peak behavior depends on the test conditions that control the development of strain localizations. Such conventional experimental devices can merely explore the micro-scale information. Advanced experimental techniques such as the photo imaging analysis (Oda and Konishi, 1974), X-ray tomography (Lee et al., 1992), wave velocity measurement (Santamarina and Cascante, 1996), magnetic resonance imaging (Ng and Wang, 2001) etc. can be used; however, they are complicated, expensive and time consuming. Moreover, not all micro-scale information can be captured using these devices and techniques. It suggests that alternative approaches are required to investigate these micro-scale responses. Numerical approach that models the discrete behaviour of granular materials such as sand can be a good alternative. DEM (Cundall and Struck, 1979) is a numerical tool that draws the attention of many researches to model the discrete behaviour of granular system in recent years. In this study, DEM is used as a numerical tool to study the macro- and micro-scale behavior of granular system for the numerical samples of different sizes. Three numerical samples were randomly generated in three different cubes of different sizes. The randomly generated samples were compressed isotropically by moving their boundaries inward. Simulations of true triaxial compression tests were conducted using these samples to investigate their behaviors at macro- and micro-scale. Digital data at macro- and micro-scale were recoded and the responses were reported.

2. NUMERICAL METHOD

DEM is a numerical method that enables one to model the discrete behavior of granular materials. It is pioneered by Cundall and Struck (Cundall and Struck, 1979). The basic idea used in DEM is simple, where each element in the model can make and break contact with its neighbors. Translational and rotational accelerations of particles in a granular assembly are computed using Newton's second law of motion. These accelerations are integrated twice with respect to time to get their displacement. Force displacement law is used to get the forces using the displacement of particles calculated earlier and the cycle continues for the next step. Translational and rotational accelerations of particles are computed as follows:

$$m\ddot{x}_i = \sum F_i \qquad i = 1,3 \tag{1}$$

$$I\ddot{\theta} = \sum M \tag{2}$$

where F_i are the force components on each particle, M is the moment, m is the mass, I is the moment of inertia, $\ddot{x}_i$ are the components of translational acceleration and $\ddot{\theta}$ is the rotational acceleration of the particle.

3. SAMPLE GENERATION AND PREPARATION

Numerical samples were generated using the open source computer code YADE (Koziki and Donze, 2008) which is based on DEM. YADE is written using C++. Three numerical samples were generated in three cubes of different sizes in such a way that no particle (sphere) can touch others. These generated samples were subjected to isotropic compression by moving their six boundaries inward. The isotropic compression was continued until the unbalanced forces became small and the porosity of the sample at the last few thousand steps became constant. The interparticle friction angles during the isotropic compression for three samples were assigned 0.01° to yield dense samples. The properties of three samples after the end of isometric compression are shown in Table 1. Figure 1(a) shows the initially generated sparse sample S1 whereas Figure 1(b) shows the isotropically compressed dense sample S1 for an example.

Table 1: Properties of three dense samples after the end of isotropic compression

Sample Designation	Size (m×m×m)	No. of Particles	Porosity
S1	0.1×0.1×0.1	2866	0.38
S2	0.125×0.125×0.125	3440	0.38
S3	0.15×0.15×0.15	3897	0.38

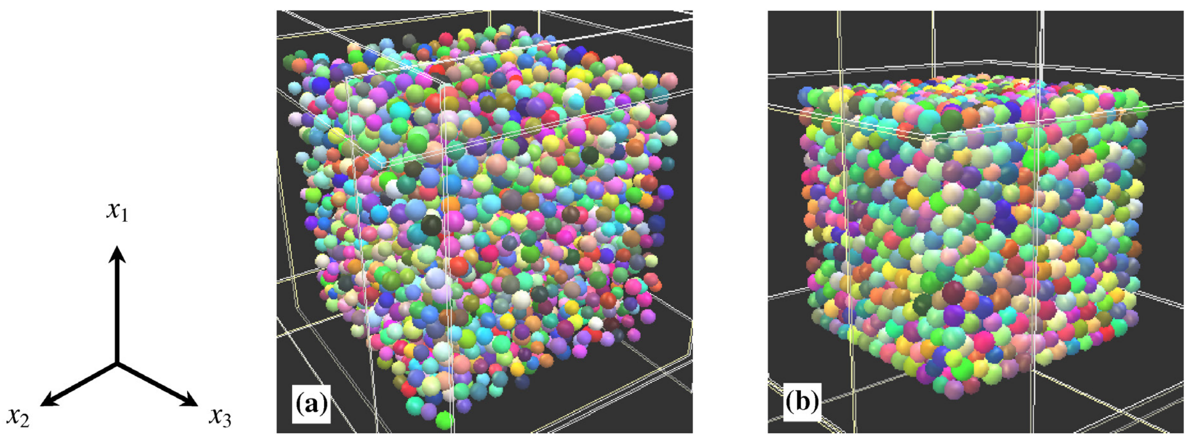

Figure 1: (a) Randomly generated sparse sample S1at the initial stage of sample preparation; (b) Sample S1 at the end of isometric compression with reference axes.

4. NUMERICAL SIMULATION

Simulation of true triaxial compression test was conducted by slowing moving the top and bottom boundaries inward the sample with a small strain rate and maintaining the confining pressures constant by continuously adjusting the position of other four boundaries of the sample. The isotropically compressed samples of different sizes were subjected to true triaxial compression in drained condition. The DEM parameters used in simulations are given in Table 2.

Table 2: Parameters used in the simulations

Parameters	Value
Box Young Modulus (N/m^2)	60×10^6
Mass density (kg/m^3)	2600
Sphere Young Modulus (N/m^2)	60×10^6
Stiffness ratio	0.50
Strain rate	0.10
Interparticle friction angle (degree)	26.5°

5. NUMERICAL RESULTS

5.1 Macro-mechanical Responses

Figure 2 shows the relationship between the deviatoric stress q [=$\sigma_1 - \sigma_3$] and axial strain ε_1, where σ_1 and σ_3 are the stresses in x_1- and x_3-direction, respectively. The deviatoric stress gradually increases with axial strain ε_1 up to peak followed by a huge strain softening. The sample size has no influence on the pre-peak behavior. However, the post-peak behavior is clearly influenced by the sample size. Note that small size sample achieves higher strength than large size sample. This behaviour is consistent with the experimental results (e.g., Jansma, 1988, Hu et al., 2011). The evolution of volumetric strain is also depicted in Figure 3. The volumetric strain is defined here as $\varepsilon_v = \varepsilon_1 + \varepsilon_2 + \varepsilon_3$, where ε_1, ε_2 and ε_3 are the strains in x_1-, x_2- and x_3-direction, respectively. The positive value of ε_v indicates compression while the negative value indicates dilation. The samples depict huge dilation after an initial small compression regardless of the sample size which is typical for a dense sample.

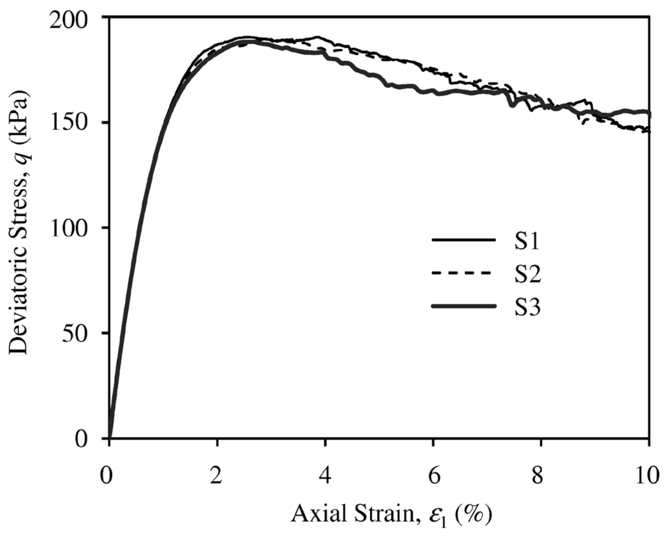

Figure 2: Stress-strain relationship for different sizes of samples

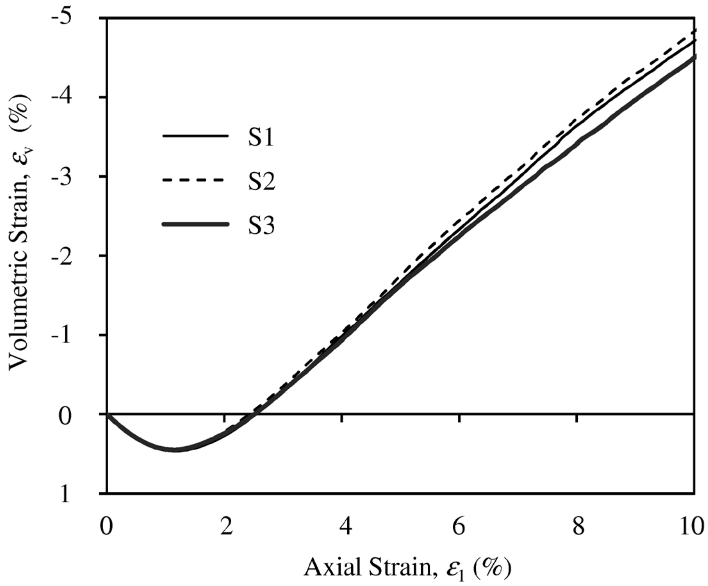

Figure 3: Volumetric strain - axial strain relationship for different sizes of samples

The relationship between the dilatancy index and axial strain is depicted in Figure 4 for samples of different sizes. The dilatancy index is defined here as

$$DI = \frac{-d\varepsilon_v}{d\varepsilon_1} \tag{3}$$

Here, $d\varepsilon_v$ is the change of volume and $d\varepsilon_1$ is the change of axial strain. The dilatancy index curve depicts that the behaviour is almost independent of the sample size. It is also noted that dilatancy index curve becomes almost straight at large strain during the simulation.

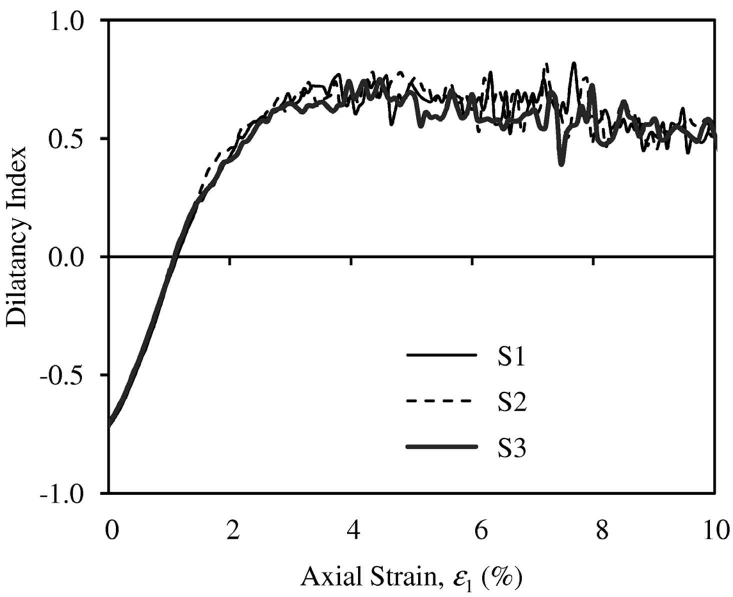

Figure 4: Relationship between dilatancy index and axial strain for samples of different sizes

5.2 Micro-scale Response

The relationship between the coordination number and axial strain is depicted in Figure 5 for different sample sizes. Coordination number is defined as

$$Z = \frac{2N_c}{N_p} \tag{4}$$

where N_c is the total number of contacts and N_p is the total number of particles. Coordination number gradually decreases with axial strain regardless of the difference in the sizes of samples. The rate of the reduction of coordination number is noteworthy at small strain range; whereas, it decreases for larger strains. Note also that the coordination number is the largest for sample of smallest size.

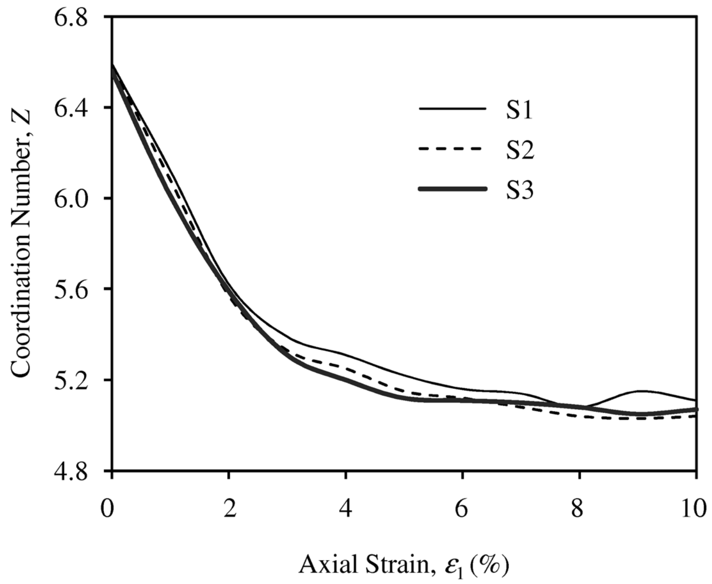

Figure 5: Relationship between coordination number and axial strain for different sample sizes

6. CONCLUSIONS

Numerical simulations were carried out using DEM to investigate the effect of sample size on both the macro- and micro-scale responses of granular materials. Samples of three different sizes were numerically generated and true triaxial tests were simulated. The digital data during the simulation were recorded at regular interval and numerical analysis was conducted. Few important points of the numerical study are summarized below:

- The simulated stress-strain behaviour is qualitatively similar to that observed in the experimental study. This reveals the versatility of the present simulation.
- The dilatancy index behaviour is almost independent of the sample size.
- Coordination number gradually decreases with axial strain regardless of the sizes of the samples.
- Evolution of coordination number is the largest for sample of smallest size.

REFERENCES

Cundall, P.A. and Strack, O.D.L. (1979). “A discrete numerical model for granular assemblies.” Geotechnique, Vol. 29(1), 47-65.

Hu, W., Dano, C., Hicher, P.Y., Le Touzo, J.Y., Derkx, F. and Merliot, E. (2011). “Effect of sample size on the behavior of granular materials”, Geotechnical Testing Journal (GTJ), Vol. 34(3), 186-197 .

Jansma, H. (1988). “Specimen size effect in cyclic loading triaxial testing”, Publication of: Delft University of Technology, Faculty of Civil Engineering, Issue Number: 7-88-201-1.

Koziki, J. , and Donze, F.V. (2008). “A new open-source software developed for numerical simulations using discrete modeling methods.”, Computer Methods in Applied Mechanics and Engineering, Vol. 197(49-50), 4429–4443.

Lee, X., Dass, W. C. and Manzione, C. W. (1992). “Characterization of granular material composit structures using computerized tomography.” In: Proc. of 9th Engng. Mechnical. Conference ASCE, New York, pp. 268–271.

Ng, T.-T., and Wang, C. (2001). “Comparison of a 3-D DEM simulation with MRI data.” Int. J. Numer. Anal. Mech. Geomech., Vol. 25(5), 497–507.

Oda, M. and Konishi, J. (1974). “Microscopic deformation mechanism of granular material in simple shear.” Soils and Foundation, Vol. 14(4), 25–38.

Santamarina, J. C. and Cascante, G. (1996). “Stress anisotropy and wave propagation: a micromechanical view.” Canadian Geotechnical J ournal, Vol. 33(5), 770–782.

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FOUNDATION SYSTEM ADOPTED TO CONSTRUCTBUILDING IN AND AROUND KUET CAMPUS OF THE BANGLADESH

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ABSTRACT

The subsoil of Southwest region of Bangladesh, especially in the surrounding of KUET campus that is situated in lowland topography with a very thick soft fine grained and organic soil deposits up to great depth. This paper describe the case studies of different foundation system adopted for the construction of some building in and around KUET campus. The building in this region are experienced very large amount of total and differential settlement. While constructed conventional shallow foundation, as a result constructed infrastructure lost their utility and abandoned in some cases. Geotechnical Engineers have been facing such difficulties for the last few decades and rate of the construction of civil infrastructure have been increased in the recent years. Based on the field experiences and to ensure the foundation stability, different types of foundation have been practiced in KUET campus such as the construction of Amar Ekushy Hall, Rokeya Hall and New Academic Building. Based on the reveled subsoil conditions of the study area, a general soil profile is adopted in this study. The adopted foundation system ranges from continuous brick footing to pile foundation. Considering the engineering soundness, safety of building and the cost, a foundation system associated with ground improvement using sand compaction piles and mat foundation over a compacted sand layer is proposed. Based on study, the soil is very soft and there is organic layer between two clayey layers. Sand cushion with mat foundation and another sand compaction pile with single column foundation is suitable. Normally pile foundation is suitable but pile length is high so cost is higher than other foundation. Brick Foundation is not suitable for this campus and around of this campus. Special attention is needed to take care for design foundation

Keywords: *Foundation, stabilization, compressive strength, laboratory, additives & capacity*

1. INTRODUCTION

Three mighty rivers the Gangas, the Brahamaputra and the Meghna and their associated tributaries contributed very much in the formation of sub soil profiles of Bangladesh which is situated in the in Bengal Basin. The South- West costal region of Bangladesh contains fine grain soil deposits with the presence of organic soil deposits with the presence of organics is due to the fact that vast of these coast regions were part of the Sundarbans, the world largest mangrove forest extends over an area 5,77,285 hactares as recorded (Razzaque and Alamgir 1999). During the geological changes in the past, some part of the Sundarbans were submerged by the weathered and sediment deposits resulting in the present peat deposits in these region. In this region peat soil is found in different layer and in different depth. Also this soil has low bearing capacity below the peat layer. For that purpose all infrastructure are settled by large amount. Due to this inherent limitation the foundation system for the construction of civil infrastructure is designed special consideration and very carefully, which leads to high cost for the preparation of sub structure in this region of Bangladesh.Bangladesh is a deltaic land, which is formed by the lower reaches of the Brahamaputra, the eastern channel of the Ganges, the Meghna and their associated tributaries. **K**hulna University of Engineering & Technology (KUET), located in the south western region of Bangladesh. The campus situated in a lowland topography with a very thick soft and organic soil deposits up to great depth. Many years ago it was the part of the biggest mangrove forest Sundarban. In course of time some part of Sundarban were submerged by sediment deposits resulting in the present peat layer in this region. Several new building have been and or are being constructed such a New Academic Building , Amar Ekushy Hall , Rokeya Hall in which different types of foundation system are considered based on the sub soil profiles , building types , purpose and the consultant preference. The selected ten constructed building of the region depicts that four different types foundation are used such that Mat foundation, Pile foundation, wall footing, Single column footing.

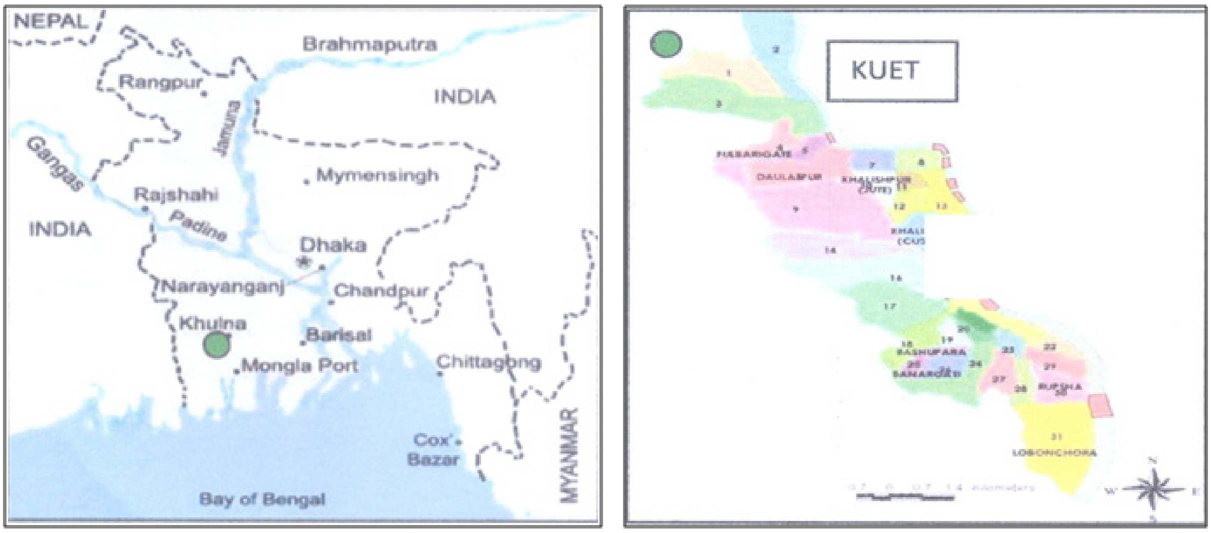


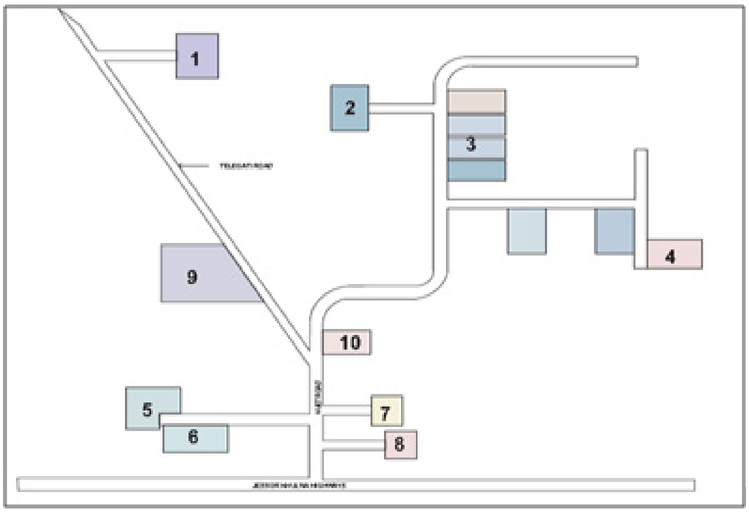
Figure 1: Location of Khulna University of Engineering & Technology (KUET) campus in Bangladesh map and the map of Khulna City

The performance of the constructed buildings are evaluated and hence discussed in this paper.Bore logs and soil profile are parts of sub soil exploration, based on which the foundation is designed. To know the nature of soil below the proposed building exactly and specially with their entire physical, chemical, and engineering properties, the sub soil exploration is done on the site. Such an exploration program is done after gathering all preliminary information such that available information, Reconnaissance, Building code requirements and Preliminary design data. Then a tentative exploration program is worked out and samples are collected at different layer so that the samples can be tested at laboratory to get different parameter. On the soil profile, the ground water table, existing construction, and the proposed structure should be also be indicated. It also be helpful if the essential engineering data, such as the standard penetration resistance, unconfined compressive strength. The arrangement of various soil layer can be best shown in the form of a geologic profile or soil profile. A geologic profile is a graphical representation of underground condition along a given line on the ground surface. In order to clearly show the various soil layers, the vertical scale is usually made large than horizontal scale. A soil profile is simple to construct.

First all boring along the profile are represented along vertical lines, with the spacing of boring drawn to conventional horizontal scale. Along each boring, the separate soil layers are shown at the correct elevations and are clearly identified. The boundaries between identical soil layers are connected to indicate the most likely at stratification. The reliability of a geologic profile as compared to the actual soil condition depends upon the nature of the ground and spacing of the boring. If the soil conditions are erratic, the arrangement of various layers between the boring may differ considerably from the interpolation. On the soil profile, the ground water level, existing construction proposed construction should also be indicated. It also helpful, if the essential engineering data, such as the standard penetration test value, unconfined compressive strength, etc are indicated profile. The main focus of this study formulated as, (1) To Know the foundation system adopted in different building; (2) To know the sub soil condition on which the building are constructed; (3) To prepare a general soil profile of our study site; (4) To justify the suitability of the foundation which could be used in different site; (5) To make the settlement calculation; (6) To analysis the foundation based on cost benefit ratio.

2. MATERIALS AND METHODOLOGY

To these attempts, 10 Nos. different types building that are situated in and outside of KUET campus are selected in this study. Soil boring are performed to know the classification of soil, N-value at different layer and Engineering properties such as unconfined compressive strength (q_u), liquid limit value(W_L), plastic limit value(W_P) and Cc value. In this study 10 different building is consider as a study materials that diagram and position are shown in Figure.2



1. Mohilla TTC
2. Amar Ehushey Hall
3. New Academic Building
4. Rokeya Hall
5. Teacher's Building
6. Moittre Naursing home
7. Fresh Clinic
8. Adjacent building of fresh clinic
9. Building beyond khan jahan ali Hall
10. Suktara Building

Figure 2: Building location in our study

Four types of foundation are used in ten different building that are mat foundation, pile foundation, wall footing and single column footing. Mat foundation are used in Amar Ekushey Hall, Sand pile are used in new Academic Building, Rokeya Hall , Moitre Nursing Home, Fresh clinic and building near Fresh Clinic, Mohila Training center and wall footing are used in suktara and bulding behind Khan Jahan Ali Hall. On the basis of soil boring, a soil profile are formed at different preselected building that are given below.

2.1 Soil profile of new academic building, KUET

The profile shows the soil characteristics up to 15m depth. The three storied building is use a academic purpose of KUET.

Table 1: Typical soil profile of New Academic Building, KUET

Depth (m)	Thickness (m)	Strata	Classification of soil	N-Value	Engineering Properties
0-3	3		lay silt with gray	4	C_c= 0.12, Gs= 2.68, q_u=61kPa W=47%, W_L=56% & W_P= 60%
3-6	3		Organic black	7	C_c= 0.52, Gs= 2.68, W=56%, q_u=39kPa, W_L=64% & W_p= 57%
6-15	9		Silty clay with gray	9	C_c=0.11,Gs=2.67,q_u=86kPa, W= 52%, W_L=58% & W_p= 79%

2.2 Soil profile of Rokeya hall, KUET

The profile shows the soil characteristics up to 15m depth. The three storied building is use as a student's Hall.

Table 2: Typical soil profile of Rokeya Hall, KUET

Depth (m)	Thickness (m)	Strata	Classification of soil	N-Value	Engineering Properties
0-3	3		ray clay with ganic	4	q_u=28kPa, W=53%, W_L=58% & W_P= 42%
3-6	3		rk gray clay with ce organic	8	q_u=29kPa, W= 53%, W_L=47% & W_p= 59%
6-15	9		Dark gray with silty clay	9	W= 53%, W_L=54% & W_p= 72%

2.3 Soil profile of amar ekushya hall, KUET

The profile shows the soil characteristics up to 15m depth. The five storied building is use as a student's Hall

Table 3: Typical soil profile of Amar Ekushya Hall, KUET

Depth (m)	Thickness (m)	Strata	Classification of soil	N-Value	Engineering Properties
0-3	3		Gray clay with organic	5	C_c= 0.16, Gs= 2.68, q_u=67kPa, W= 51%, W_L=48% & W_P= 42%
3-6	3		Dark gray clay with trace organic	6	q_u=28kPa, W= 66%, W_L=59% & W_p= 57%
6-15	9		Dark gray with silty clay	9	W= 53%, W_L=57% & W_p= 73%

2.4 Soil profile of mohilla technical training center

The profile shows the soil characteristics up to 15m depth. The three storied building is use as a office and classroom.

Table 4: Typical soil profile Mohilla Technical Training Center

Depth (m)	Thickness (m)	Strata	Classification of soil	N-Value	Engineering Properties
0-3	3		rganic clay , ark gray	5	C_c= 0.16, Gs= 2.72,q_u=67kPa, W=58%, W_L=50% & W_P= 42%
3-6	3		lty clay with composed mber, dark Gray	7	W_p= 61%
6-15	9		Dark gray with silty clay	9	W= 52%, W_L=57% & W_p= 67%

2.5 Soil profile of maitre nursing home, Fulbarigate

The profile shows the soil characteristics up to 15m depth. The three storied building is use as a hospital.

Table 5: Typical soil profile Maitre Nursing Home, Fulbarigate

Depth (m)	Thickness (m)	Strata	Classification of soil	N-Value	Engineering Properties
0-3	3		rk gray with silty y	5	C_c= 0.168, Gs= 2.67, q_u=67kPa, W=46%, W_L=55% & W_P= 45%
3-6	3		rk gray with anic	6	Gs= 2.68, W=59%, W_L=55% & W_p= 74%
6-15	9		Dark gray with silty clay	5	Gs= 2.67,W= 58% & W_p= 70%

2.6 Soil profile of near maitre nursing home, Fulbarigate

The profile shows the soil characteristics up to 15m depth. The four storied building is use as a residential building.

Table 6: Typical soil profile near maitre nursing home, Fulbarigate

Depth (m)	Thickness (m)	Strata	Classification of soil	N-Value	Engineering Properties
0-3	3		k gray with silty y	5	C_c= 0.168, Gs= 2.68,W=52%, W_L=58% & W_P= 48%
3-6	3		k gray with silty y	8	Gs= 2.69, W=63%, W_L=49% & W_p= 64%
6-15	9		Dark gray with silty clay	9	C_c=0.168,Gs=2.66,W= 60%, W_L=58% & W_p= 74%

2.7 Soil profile of fresh clinic, fulbarigate

The profile shows the soil characteristics up to 15m depth. The five storied building is use as a clinic

Depth (m)	Thickness (m)	Strata	Classification of soil	N-Value	Engineering Properties
0-3	3		Clay silt with gray	5	C_c= 0.15, Gs= 2.71, q_u=55kPa W=38%, W_L=42% & W_P= 30%
3-6	3		Organic clay with dark gray	8	Gs= 2.69, W=63%, q_u=36kPa W_L=49% & W_p= 64%
6-15	9		Silty clay with gray	9	C_c=0.24,Gs=2.67,q_u=79kPa, W= 53%, W_L=45% & W_p= 77%

2.8 2.8 Soil profile of adjacent building fresh clinic, Fulbarigate

The profile shows the soil characteristics up to 15m depth. The five storied building is use as a residential building

Table 8: Typical soil profile of adjacent building of fresh clinic, Fulbarigate

Depth (m)	Thickness (m)	Strata	Classification of soil	N-Value	Engineering Properties
0-3	3		ay silt with gray	5	C_c= 0.15, Gs= 2.71, q_u=55kPa W=48%, W_L=62% & W_P= 45%
3-6	3		rganic clay with rk gray	8	Gs= 2.72, W=64%, q_u=37kPa, W_L=55% & W_p= 55%
6-15	9		Silty clay with gray	9	C_c=0.17,Gs=2.67,q_u=75kPa, W= 60% & W_L=58%

2.9 Soil profile of building behind khan jahan ali hall

The profile shows the soil characteristics up to 15m depth. The for storied building is use as a residential building

Table 9: Typical soil profile of building behind khan jahan ali hall

Depth (m)	Thickness (m)	Strata	Classification of soil	N-Value	Engineering Properties
0-3	3		ay silt with trace	4	C_c= 0.12, Gs= 2.68, q_u=61kPa W=47%, W_L=56% & W_P= 60%
3-6	3		Organic black	7	C_c= 0.52, Gs= 2.68, W=56%, q_u=39kPa, W_L=64% & W_p= 57%
6-15	9		Silty clay with gray	9	C_c=0.11,Gs=2.67,q_u=86kPa, W= 52%, W_L=58% & W_p= 79%

2.10 Soil profile of building suktara near khan jahan ali hall

The profile shows the soil characteristics up to 15m depth. The four storied building is use as a residential building.

Table 10: Typical soil profile of building suktara near khan jahan ali hall

Depth (m)	Thickness (m)	Strata	Classification of soil	N-Value	Engineering Properties
0-3	3		y silt with gray	4	Gs= 2.71, q_u=55kPa, W=52%, W_L=59% & W_P= 60%
3-6	3		Organic black	6	C_c= 0.51, Gs= 2.72, W=64%, q_u=39kPa, W_L=64% & W_p= 65%
6-15	9		Clayey silt with gray	7	C_c=0.19,Gs=2.71,q_u=87kPa, W= 57%, W_L=52% & W_p= 61%

3. RESULT AND DISCUSSIONS

In south –West region of Bangladesh, civil infrastructure suffers for very large amount differential settlement. In general soil profile, it is seen that there is an organic layer between two clay layers. The organic layers is very much compressed and eventually contributes in large settlement secondary consolidation. If mat foundation is used then excessive settlement occure. In this case pile may be used. But due to the presence of organic layer, there is a possibility of negative skin friction. To reduce this negative skin friction, it is necessary to increase the diameter and length of pile foundation. Which requires excessive cost. So ground improvement can be a good alternative to solve geotechnical engineering problem. Sand compaction pile with a granular soil layer over it can be simplest ground improvement techniques. So after ground improvement mat foundation may be provided to get the best service. Although use of pile foundation is safe for the super structure but other type of foundation were used for low cost. Sometimes ground improvement was done to improve the geotechnical engineering properties of the soil. In this case geo-textile method was used to improve the ground condition. Pile foundation is used when the super structure with the water even with large number of piles neither the necessary compression nor sufficient friction can be obtained and farmer layers exists only at a depth which is unreachable , a heavy building can safely founded by sinking it partly into the ground so that it actually float.

4. CONCLUSION

- Large settlement occurs at the building while Mat foundation is used without soil improvement
- In our study site, the soil is very soft and there is organic layer between two clayey layers
- Two type of foundation is recommended for the study site. One is sand cushion with mat foundation and another is sand compaction pile with single column footing.
- Normally pile foundation is suitable because it is less or zero settlement. But it pile length is high so cost is higher to the other foundation
- Brick foundation is not suitable for this campus
- Special attention to take care for design of foundation in South-West region of the Banglades

REFERENCES

Bureue of research, testing and consultancy (BRTC). (1988). "Soil investigation and constructing building foundation report of proposed Khulna University of selected Gollamary Area". Department of Civil Engineering, Bangladesh University of Engineering and Technology.

SAAR Associates limited, (1992). "Consultants reports about the design and construction of academic building - 1 at Khulna University". Khulna, Bangladesh

Shaheedullah Associate Limited, (1992). "Consultants reports about the design and construction of academic building -1 at Khulna University". Khulna, Bangladesh

Mukherjee, B.B.(1992). "Quaternary pollen analysis as a possible indicator of prehistoric agriculture in the deltaic part of west bangle". J. Playnol, Vol. 8: 144-151

Sen, P. and Bananerjee, M. (1984). "On the occurrence of reptilian remnants in the peat bed of Barrackpore. 24-Parganas,West Bengal" Paleobot. Soc. Ind. Vol.29:47-51

Vishnu Mitre and Gupta, H. P.(1972). "Pollen analysis study of quaternary deposit in the Bangla" Basin. Paleobotanist. vol.19: 297-306

Razzaque, M.A and Alamgir, M. (1999). "Long term settlement observation of a building in a peat deposits of Bangladesh" International Conference on Civil & Environmental Engineering- New Frontiers and Challenges. Bangkok. Vol.2: III-85 to IV-94

DETERMINATION OF RELATIVE DENSITY OF A GRANULAR ASSEMBLY IN DEM BASED MODELING

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ABSTRACT

In this paper, a numerical procedure is used to determine the relative density of a numerical sample using DEM considering the confining pressure dependency. True triaxial compression tests were simulated by using the isotropically compressed cubic samples under the confining pressures of 50, 100 and 200 kPa by using the three-dimensional DEM. The loosest state of the numerical sample is determined by using increasing interparticle friction angles until the void ratios at successive isotropic compressions become almost constant. This constant void ratio is considered to be the maximum void ratio. The constant values of the void ratios are examined for three different confining pressures. It is noted that both the minimum and maximum void ratios of the numerical sample are confining pressure dependent. It is concluded that the minimum and maximum void ratios should be separately determined for each confining pressure to compute the relative density of the numerical sample. Two additional numerical samples each having the relative density of 82.6% and 38.9% were prepared considering 100 kPa confining pressure. True triaxial compression tests were simulated using DEM. It is noted that the numerical results are qualitatively consistent to that observed in the experimental studies.

Keywords: Relative density, confining pressure, simulation, numerical sample, void ratio

1. INTRODUCTION

Granular materials, such as sand, are discrete in nature and their behaviors at macro-scale depend on their behaviors at micro-scale. The discrete media is highly influenced by the state of density. Such discrete system is often modeled using the Discrete Element Method (DEM) pioneered by Cundall and Struck (1979). However, the relative state of density of a numerical sample in DEM based studies is usually described by the porosity or by the void ratio instead of the relative density. This is because; the loosest state of a numerical sample cannot be easily defined in DEM based modeling. If different samples having different aspect ratios of particles are prepared, it is not guaranteed that the relative density remains same for each sample even though the void ratio after the end of isotropic compression remains same. This is because; the maximum and minimum void ratios of different samples having different aspect ratios of particles may be different. This suggests that the use of relative density is the best choice to define the state of density and perform the comparative studies in DEM based modeling. However, it is hardly carried out in DEM based numerical studies due to the difficulty in defining the loosest state of the numerical sample. There are very few numerical studies that attempted the use of relative density in DEM modeling. For example, Salot et al. (2009) proposed a numerical approach to determine the relative density of a numerical assembly by radius expansion method where the radii of particles are slowly increased until the system reaches a given mean pressure on boundaries to get the minimum density state. In the present study, a simple numerical technique, different from the previous one, is adopted to compute the relative density of a numerical assembly by compressing the samples isotropically using increasing interparticle friction angle until the successive void ratios of the successive isotropic compressions remain constant. This state is defined as the loosest state. Since confining pressures also have influence on the determination of the minimum and maximum void ratios of samples, the effect of confining pressures is also considered to define the maximum and minimum void ratios of the samples. The maximum and minimum void ratios of the samples are determined using the above technique and the influence of confining pressures is carefully examined. Using the definition of relative density of the present study considering confining pressure dependency, two additional numerical samples were prepared for relative densities of 82.6% and 38.9%. These numerical samples were subjected to shear with a confining pressure of 100 kPa and the simulated results are qualitatively validated with the experimental results to examine the applicability of the present technique with confining pressure dependency.

2. METHODOLOGY

DEM is used in the present simulation. In DEM, each particle is considered as an element which can make and break contact with its neighbor. The particles can translate and rotate when the system is subjected to loading. Newton's second law of motion is employed to calculate the acceleration of the particle. This acceleration is integrated twice with respect to time to get the displacement. The displacement is used in the force displacement law to get the force and the cycle continues for the next step. Readers are referred to Cundall and Strack (1979) for further details of DEM. The translational and rotational accelerations of particles are computed as follows:

$$m\ddot{x}_i = \sum F_i \qquad i = 1,3 \tag{1}$$

$$I\ddot{\theta} = \sum M \tag{2}$$

where F_i are the force components, M is the moment, m is the mass, I is the moment of inertia, $\ddot{x}_i$ are the components of translational acceleration and $\ddot{\theta}$ is the rotational acceleration of the particle.

3. ISOTROPIC SAMPLE PREPARATION

A sparse sample was generated using the computer code YADE (Koziki and Donze, 2008) in a cube for each confining pressure such that no particle in the cube could touch the others. In this study, spheres were chosen as particles because of relatively less computational cost during simulations. Around 4000 particles were generated randomly in the cube for each confining pressure and compressed isotropically by moving the six rigid boundaries of the sample inward with the given confining pressure (50 or 100 or 200 kPa, whichever applicable). The rate of compression was kept fairly small to reduce the unbalanced forces during the isotropic compression. The isotropic compression continued for several thousand steps until the confining pressure reached the desired level (50 or 100 or 200 kPa, whichever applicable) and the porosity of the sample became constant for the last few thousand steps. The parameters used in the study are shown in Table 1. An isotropically compressed sample is depicted in Figure 1 (with reference axes) for the isotropic compression of 100 kPa confining pressure with zero interparticle friction angle. During the isotropic compression, interparticle friction angle was varied as required. During the simulation, interparticle friction angle was set to 26.5°.

Table 1: Parameters used in the simulations

Parameters	Value
Young Modulus (N/m^2)	60×10^6
Mass density (kg/m^3)	2600
Ratio of shear to normal stiffness	0.50
Strain rate	0.1

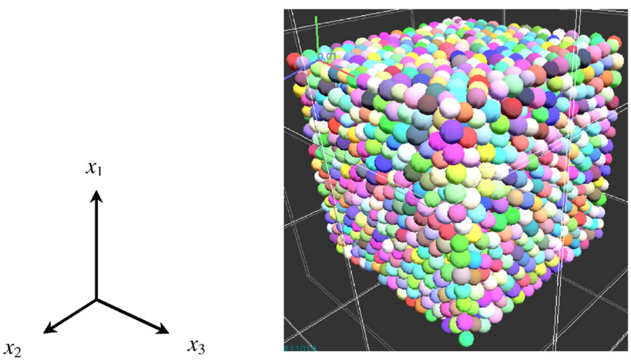

Figure 1: Isotropically compressed sample

4. TECHNIQUE TO FIND DENSEST AND LOOSEST STATE

The numerical technique depends totally on the application of interparticle friction angle during the isotropic compression to control the state of density of samples. The steps used to define the densest and loosest states of samples are reported below:

- The interparticle friction angle is assigned zero during the isotropic compression to obtain the densest state of the sample for all confining pressures. The void ratio obtained at the end of isotropic compression is regarded as the minimum void ratio ($e_{\min}$). This void ratio is considered to be the minimum one, because the particles have no restriction to move and slide due to assigning zero interparticle friction angles.

- To find the loosest state, the value of interparticle friction angle in the successive isotropic compression tests is gradually increased and the void ratio is recorded at the end of isotropic compression. When the void ratio does not chance or becomes almost constant even though the interparticle friction angles increase for the two successive isotropic compression tests, it can be presumed that the loosest state of the sample for the given confining pressure is reached. The void ratio obtained at the end of isotropic compression at this stage is regarded as the maximum void ratio ($e_{\max}$). Same technique is used for all the confining pressures.

Figure 2 depicts the plot of void ratio at the end of isotropic compression against the interparticle friction angle for different confining pressures. The void ratio at the end of isotropic compression increases with interparticle friction angle and at higher friction angles, the void ratios become almost constant. This indicates that the void ratio at higher interparticle friction angles reaches a constant or steady state. This constant void ratio is considered to be the loosest state and the void ratio at this state is regarded as the maximum void ratio $e_{\max}$. The void ratio at zero interparticle friction angle is considered as the densest state since the particles can best be compressed at zero interparticle friction. Note that the relationship is dependent on the confining pressures. The void ratio at zero interparticle friction angle for different confining pressures is not same. This indicates that the densest state is also dependent on the confining pressure. Since the void ratios at the densest state are confining pressure dependent, it is suggested that the minimum and maximum void ratios should be separately determined for each confining pressure to compute the relative density of the numerical sample.

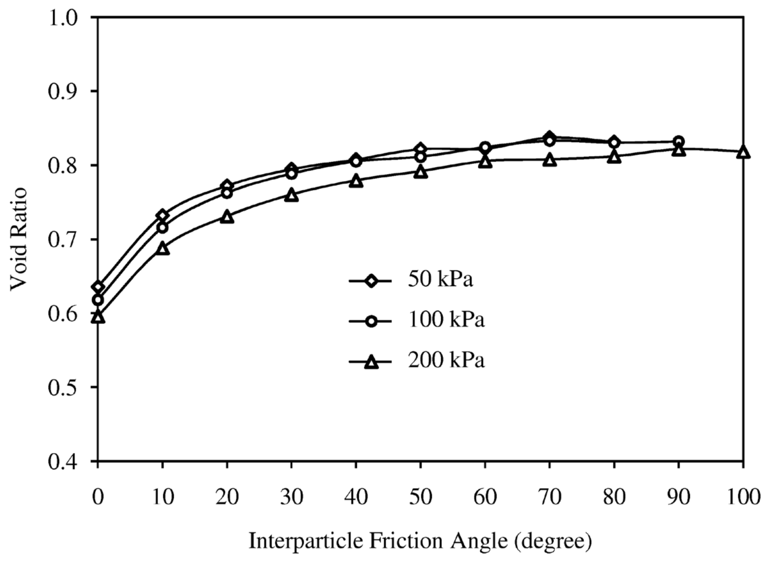

Figure 2: Relationship between the void ratio at the end of isotropic compression for three different confining pressures and the interparticle friction angle

5. SIMULATION OF TRUE TRIAXIAL TESTS

Following the technique described in the above section, two additional numerical samples (dense and loose) were prepared for 100 kPa confining pressure, the relative densities $D_r = (e_{max} - e)/(e_{max} - e_{min})$ of which are 82.6% and 38.9%, respectively. Here, e_{max} is the maximum void ratio at the loosest state, e_{min} is the minimum void ratio at the densest state for a given confining pressure. True triaxial compression tests were simulated by moving the top and bottom boundaries inward the sample with a very small strain rate and maintaining the confining pressure 100 kPa by continuously adjusting the position of the lateral boundaries. The simulations were conducted to examine whether the present definition works or not

Figure 3 depicts the relationship between the stress ratio σ_1/σ_3 and the axial strain ε_1 for two different relative densities of the samples under 100 kPa confining pressure. Here, σ_1 and σ_3 are the stresses in x_1- and x_3- directions, respectively. For dense sample (D_r =82.6%), the stress ratio increases gradually till the peak followed by a huge strain softening. This is the typical behavior for dense sand carried out in the laboratory experiment. The loose sample (D_r =38.9%), on the other hand, increases gradually till the residual state. This behavior is also typical for loose sand carried out in the laboratory experiment. Note also that the stress ratios are approaching to the critical state at large strain. The behaviors discussed above are similar qualitatively to the experimental results for dense and loose sand (e.g., Al-Hussaini, 1973; Baladi et al., 1988; Terzaghi et al., 1996). This similarity suggests that the numerical approach, applied here considering the confining pressure dependency, works.

Figure 4 depicts the relationship between the volumetric strain ε_v and the axial strain ε_1 for two different relative densities of the samples under 100 kPa confining pressure. The volumetric strain is defined here as $\varepsilon_v = \varepsilon_1 + \varepsilon_2 + \varepsilon_3$, where ε_1, ε_2 and ε_3 are the strains in x_1-, x_2- and x_3- directions, respectively. The positive value of ε_v is considered as compression, while the negative value is considered as dilation. Dense sample (D_r =82.6%) shows initial compression which is followed by huge dilation. By contrast, loose sample (D_r =38.9%) shows compressive behavior and at large strain, it shows little dilation. The behaviors are typical for dense and loose samples, observed in the experimental studies. The behaviors discussed above are similar qualitatively to the experimental results for dense and loose sand (e.g., Al-Hussaini, 1973; Baladi et al., 1988; Terzaghi et al., 1996). This similarity also suggests that the numerical approach, applied here considering the confining pressure dependency, works.

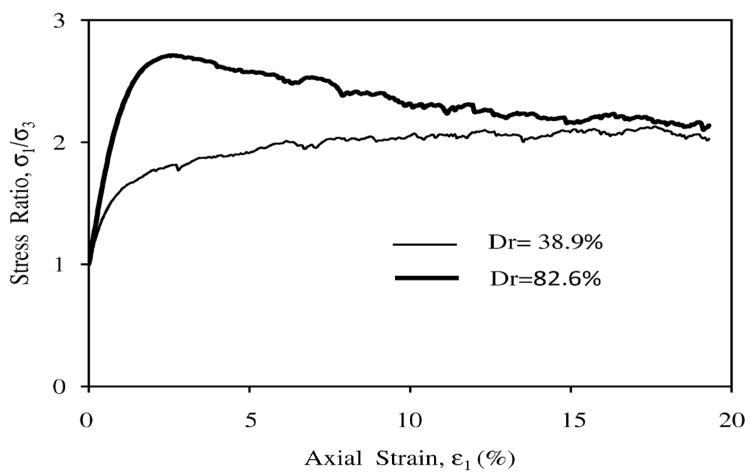

Figure 3: Stress-strain relationship for different relative densities of samples under 100 kPa confining pressures

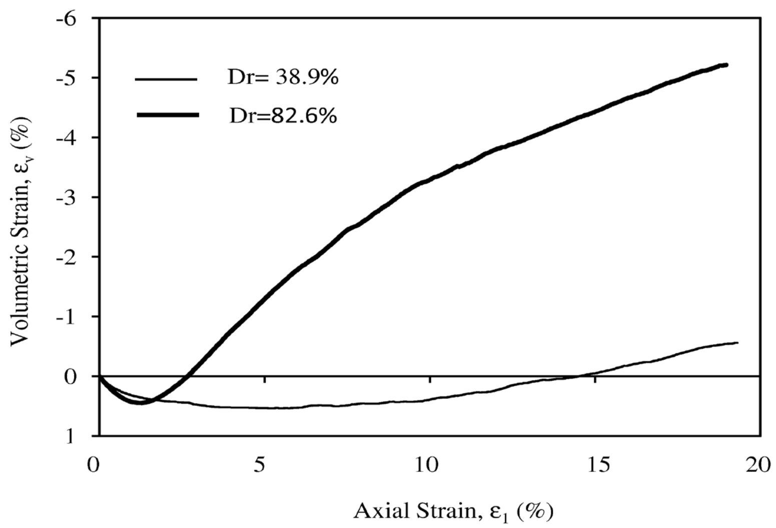

Figure 4: Relationship between volumetric strain and axial strain for different relative densities of samples under 100 kPa confining pressures

6. CONCLUSIONS

A numerical procedure is applied in the present study to determine the relative density of a granular sample using DEM considering the confining pressure dependency. Minimum and maximum void ratios are determined and their applicability is shown through two true triaxial compression tests simulated under the confining pressure of 100 kPa using the three-dimensional DEM. Few important points of the numerical study can be summarized as follows:

- The loosest state of the numerical sample is determined by using increasing interparticle friction angles until the void ratio at successive simulations is almost constant.
- This densest state of the numerical sample is determined by using the zero interparticle friction angle.
- Both the minimum and maximum void ratios of the numerical sample are confining pressure dependent.
- The minimum and maximum void ratios should be separately determined for each confining pressure to compute the relative density of the numerical sample

REFERENCES

Al-Hussaini, M. M. (1973). “Influence of relative density on the strength and deformation of sand under plane strain condition.” Evaluation of relative density and its role in geotechnical project involving cohesionless soil, ASTM, STP 523, ASTM,332-347

Baladi, G. Y. and Wu, T. T. H. (1988). “ Interpretation of Triaxial Test Results of Results of Cohesionless Soils: A new Model. Advanced Triaxial Testing of Soil and Rock, ASTM STP 977. Robert T., Ronald C. and Marshall L. Silver, Edis, ASTM, Philadelphia, pp. 567-581.

Cundall, P.A., and Strack, O.D.L. (1979). “A discrete numerical model for granular assemblies.” Geotechnique, Vol. 29(1), 47-65.

Koziki, J. , and Donze, F.V. (2008). “A new open-source software developed for numerical simulations using discrete modeling methods.”, Computer Methods in Applied Mechanics and Engineering, Vol.197(49-50), 4429–4443.

Salot, C., Gotteland, P., and Villard, P. (2009). “Influence of relative density on granular materials behavior: DEM simulations of triaxial tests.”, Granular Matter, 11(4), 221-236.

Terzaghi, K., Peck, R. B. and Mesri, G. (1996). “Soil Mechanics in Engineering Practice”, 3rd Edition, pp. 137-138, Wiley & Sons.

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FINITE ELEMENT BASED DESIGN OF SUPPORT SYSTEM FOR THE DEEP EXCAVATION OF AN IMPOUNDING RESERVOIR: A CASE STUDY

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ABSTRACT

Deep excavations are one of the adventurous tasks taken up by geotechnical engineers. Proficient and secure design of deep excavations is a challenging task by itself. In this problem, deformations are consequential to stress release and redistribution in deep excavations and stability controls the design of the support systems. Usually, deformation is the supervision of deep excavations at field that aid to attain economy and safety. It also aids in prediction of the ground settlement on adjacent structures. The objective of this paper is to discuss the design of sheet pile, supporting structures and challenges faced in the construction of a deep impounding reservoir for a water supply project in the south-western part of Bangladesh.

Keywords: *Deep excavation, Sheet pile, Finite Element Analysis, PLAXIS*

1. INTRODUCTION

Construction of deep reservoirs, basements, subways and service tunnels require deep excavations eventually making them contingent in the construction activities. During excavation, an in situ wall system is often constructed to provide stability and to minimize movements of the adjacent ground. Excavation is one of the most hazardous construction operations as the behavior is multifaceted and their failures are rapid. The effects are caused due to the decrease in vertical stress and loss in lateral support and hence requires the examination of field performance and monitoring [Burland and Hancock, 1977; O'Rourke, 1981; Finno et. al., 1989]. Stability and deformation are the weightage factors in the performance of a deep foundation. If the factor of safety is small, strains are small and in turn the ground movements are small. The deformations damage the adjacent structures like buildings and utilities. Severity of this damage depends on the pattern and movements around the excavation. Movements associated with excavations are related to a number of factors including: base stability, soil type, soils responding to off-loading and pore-water pressure changes, wall type and its system stiffness, construction procedures and workmanship [Peck, 1969; Goldberg et. al., 1976; Clough and O'Rourke, 1990]. In order to ensure a successful excavation work, the behaviors of the wall and the adjacent ground must be considered during the design phase. It is difficult to make a direct and quantitative analysis of ground movements associated with excavation support since the total deformation is a complex interaction of the above factors. Though stability is easy to analyze using equilibrium calculations, deformations are difficult to predict. The finite element method is one of the powerful numerical methods available to date to predict the ground movement pattern in and around an excavation. In the last 3 to 4 decades several authors [Borja, 1990; Schweiger and Freiseder, 1994; Tabrizi et. al., 1995; Ou et. al. 1996; Lee et. al., 1998] used the finite element method to predict or to back analyze the performance of an excavation. Since then there is a lot of progress in understanding the material behavior of soils, and the application of the finite element method to geotechnical problems. In the seventies and eighties, the analysis of excavations using the finite element method was mostly made using undrained, linear or non-linear, elastic or elasto-plastic soil behaviours, but nowadays sophisticated programs are available for both drained and undrained analysis. Several finite element programs that are written specifically for geotechnical purposes are also commercially available.

This paper describes the design process of the support system for a deep excavation in the construction of a deep impounding reservoir of water supply project using the finite element method.

2. PROJECT DETAILS

Khulna is one of the largest cities of Bangladesh. It is located in the south-western part of Bangladesh and situated on the banks of Rupsha and Bhairab River. To achieve the goal for supplying 11 crore liter pure drinking water daily in Khulna city, the following activities will be undertaken under the project. These include rehabilitation of deep tube wells and expansion of water distribution network, establishment of water distribution pipeline, construction of surface water treatment plant, distribution reservoir and overhead tanks, strengthening corporate management system of Khulna WASA and related other activities. Under the project, the water distribution system will be extended and water will be delivered efficiently in Khulna city. An additional 490,000 persons will have a new connection to the piped network, and 220,000 persons who already have a connection will get benefit from the improved services. Of the estimated project cost of $363.60 million, Japan International Cooperation Agency (JICA) will provide about $ 184 million while the rest of $ 104.60 million will be provided by the government apart from ADB financing. The project will develop a sustainable water supply system in Khulna city, which relies entirely on groundwater. It will introduce surface water as the main water source for sustainable water resource management in Khulna city. The project adopts a climate-proof design, adapting to an expected increase in the salinity of the river water as a result of sea level rise.

The sub-soil of this region is composed of sand, silt and clay in various proportions with small amount of coarse sand, which is classified into seven litho-stratigraphic units from base to top. Stratigraphic cross-sections and panel diagram through the Khulna area indicate the presence of seven sedimentary cycles, each cycle resembling fining upward sequence. Complexes of channels of fluvial/tidal origin, natural levees, bars, swamps and plains like floodplain, deltaic plains, estuarine plains or coastal plain constitute the Khulna area. Channels (tidal as well as fluvial), natural levee, flood plain, flood basin, ox-bow lake, abandoned channels, bars, swamps/flood basins and estuarine plain have been recognized as geomorphological units within the KCC area. Of these the area occupied by the natural levee, flood plain and bars are ranked high for future urban development.

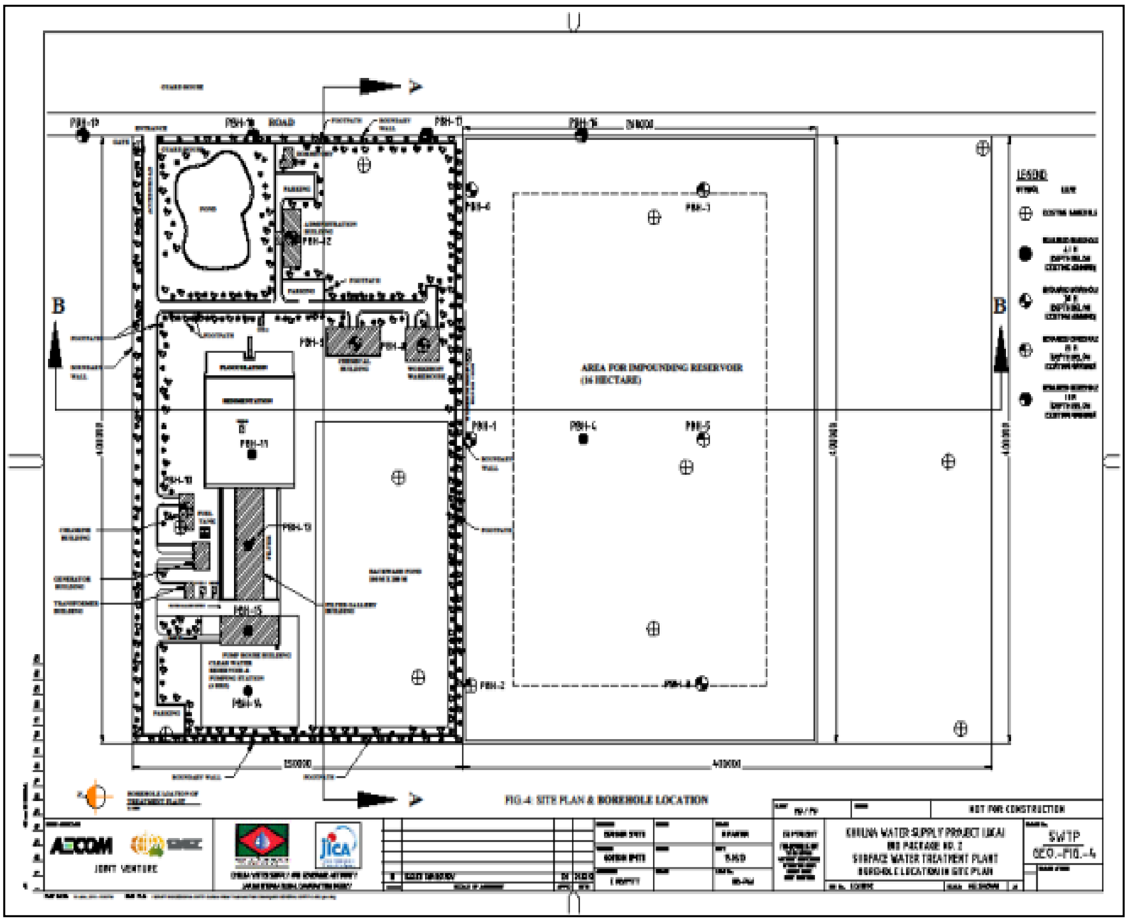

Figure 13: SWTP site layout plan with borehole location.

Khulna WASA (Water and Sewerage Authority) established on 2 March 2008 is the authority for the water supply and sanitation services in Khulna city and recently has taken a project for water supply in the city. The project area of KWASA is located in this city. The Bhairab on northern side, Rupsa River in the middle part and Pasur on the southern side flows along eastern margin of the city and Mayur on the northern side and Hatia River on the southern side flow along the western side of the city (Fig.1). The investigated area falls within the western part of Faridpur Trough of Bengal Foredeep. The trough is filled with Tertiary and Quaternary sand and clay rich sediments with few coarse sand beds. The geotechnical investigation of the project area was conducted to prepare a geological report. Figure 1 shows the boreholes locations of the Surface Water Treatment Plant (SWTP) area. Figure 2 (a, b) shows the soil profile through "AA" and "BB" sections of the site.

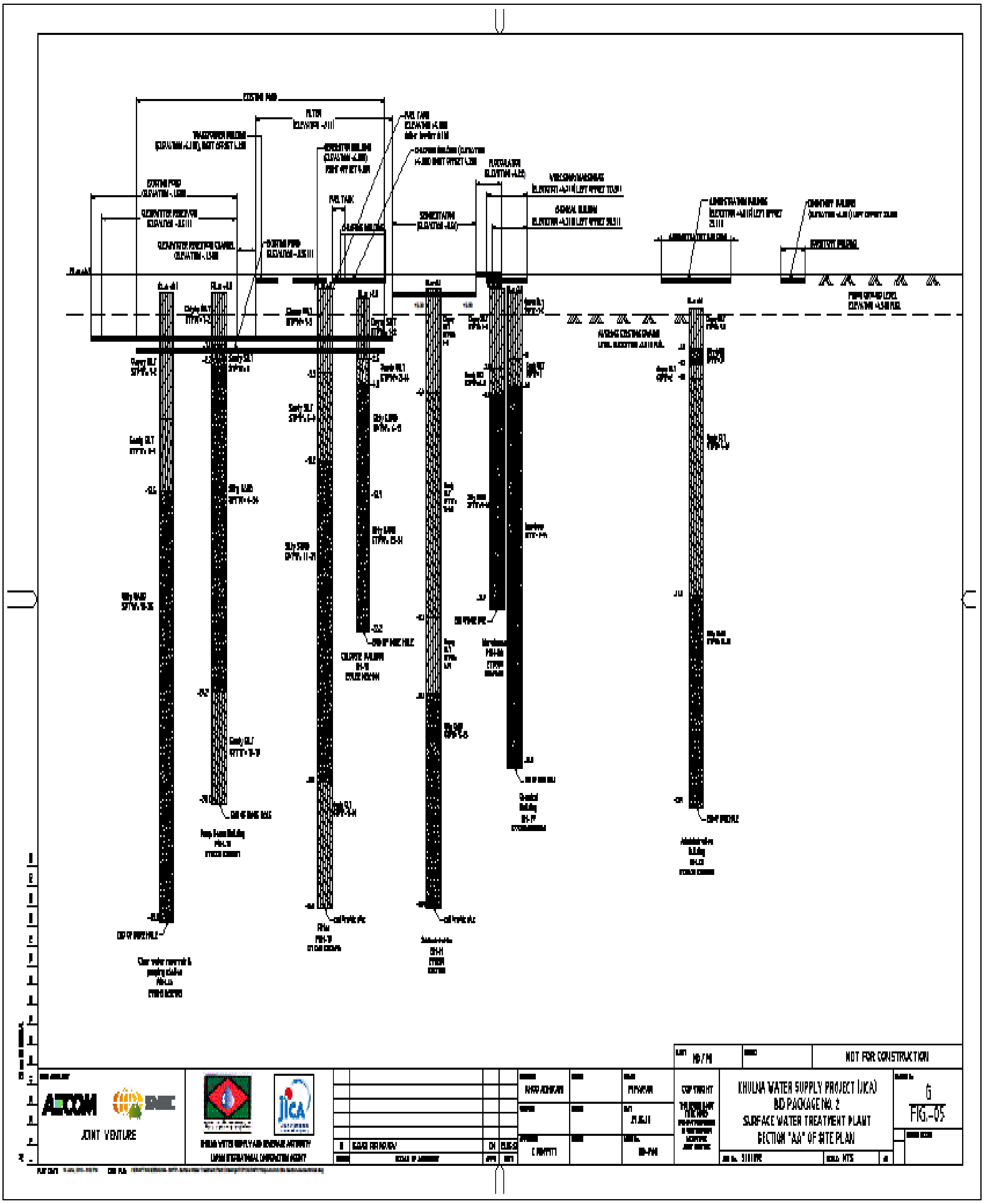

Figure 2a: Soil Profile (Section AA)

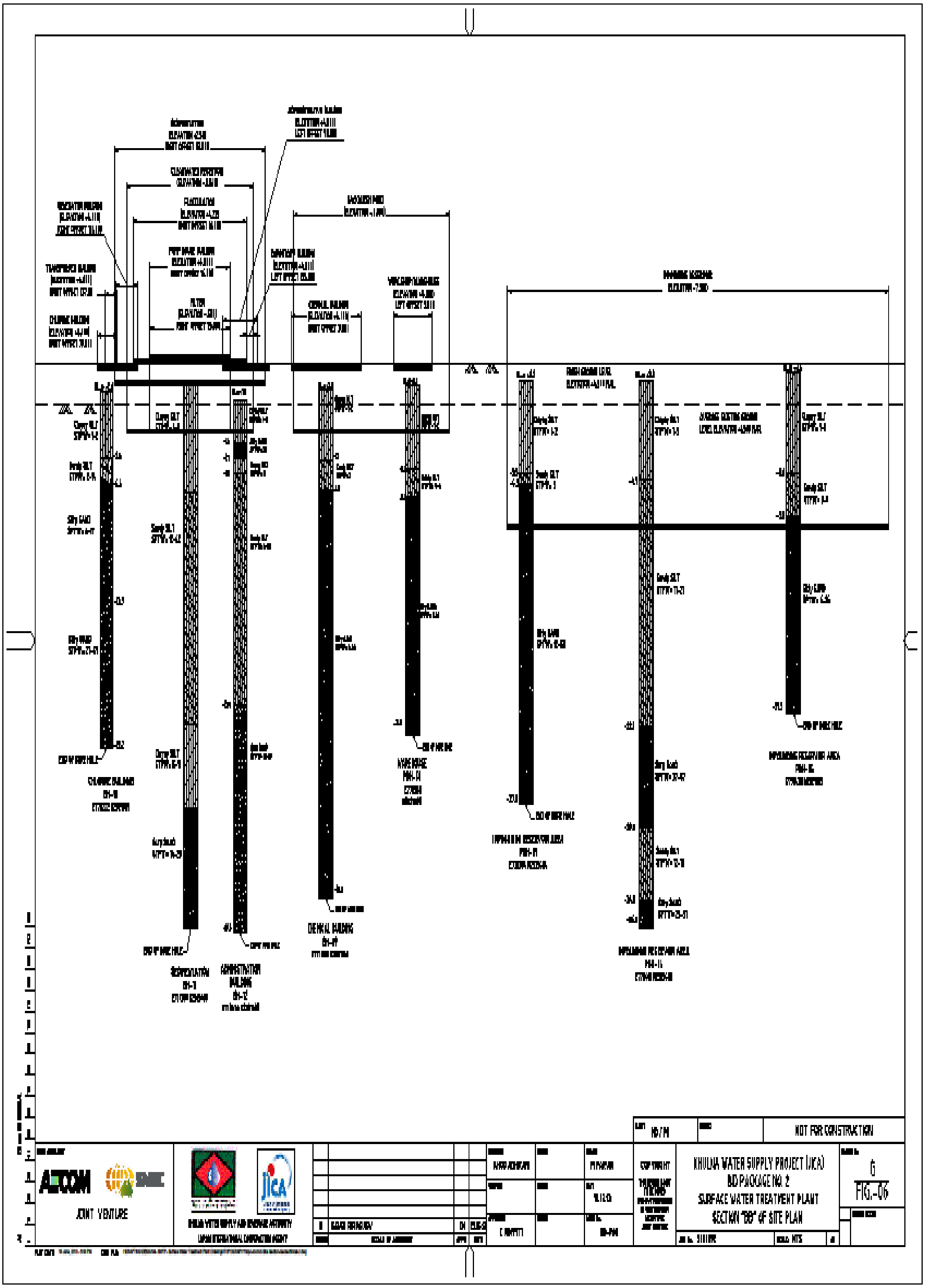

Figure 2b: Soil Profile (Section BB)

The subsoil of the proposed SWTP site is composed of sea deposit and the river basin deposit. The sub-soil strata from 4 to 8.2m depth is quite variable with irregular interlayered layers/strata of clayey/silty SAND and sandy CLAY/SILT. Below 10m depth and between 15.0 to 25.0m depth b.e.g.l. a load bearing stratum of silty fine SAND layer encountered. Below 30m depth a medium stiff to stiff layer of sandy SILT layer encountered until 40m depth (the maximum depth of the borehole) below existing ground level (+1.500m MSL). The general recommendations for the supporting structures at the SWTP area for the excavation of the impounding reservoir are shown in the Figure 3 (a, b).

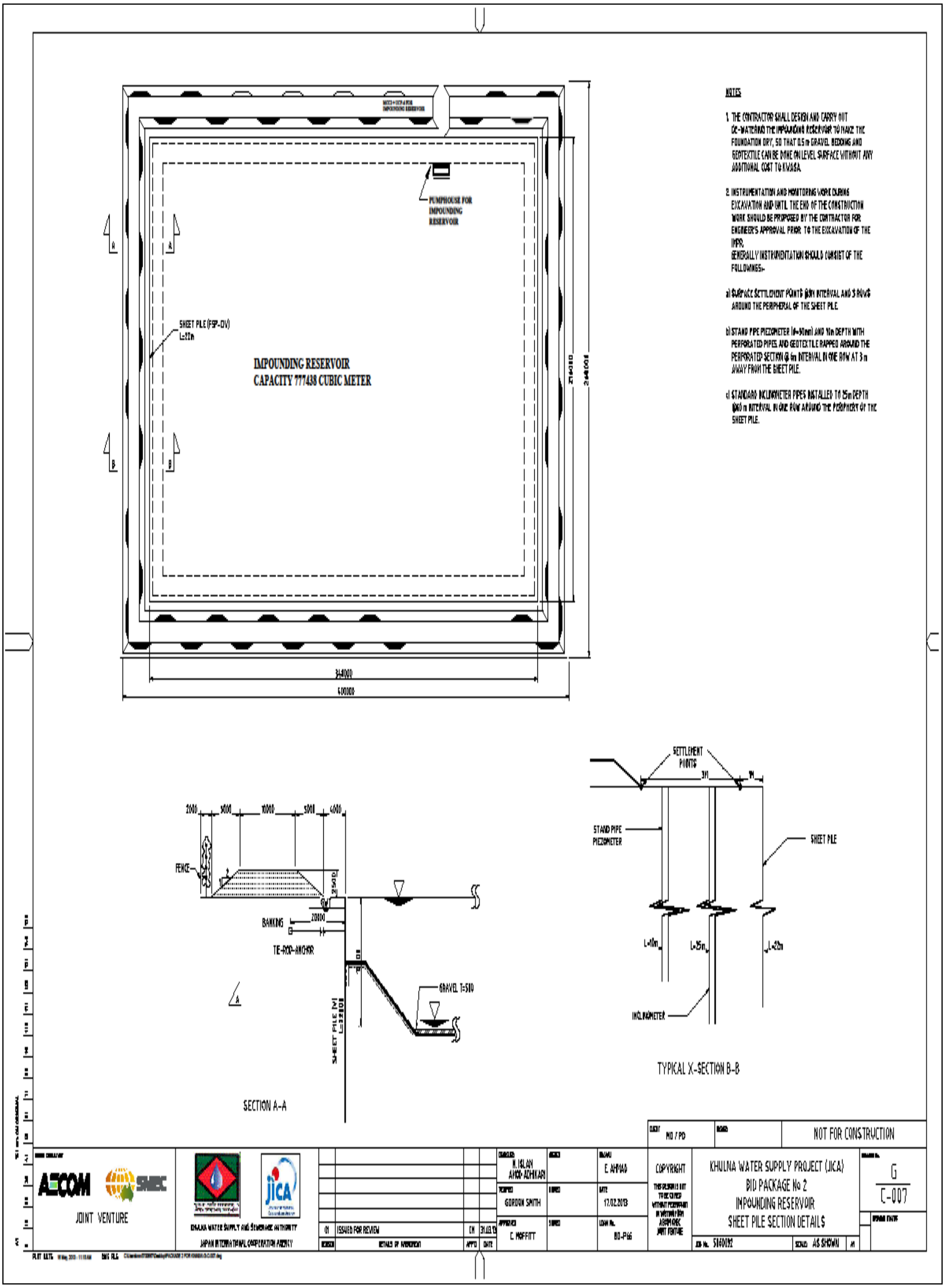

Figure 3a: Sheet pile section details

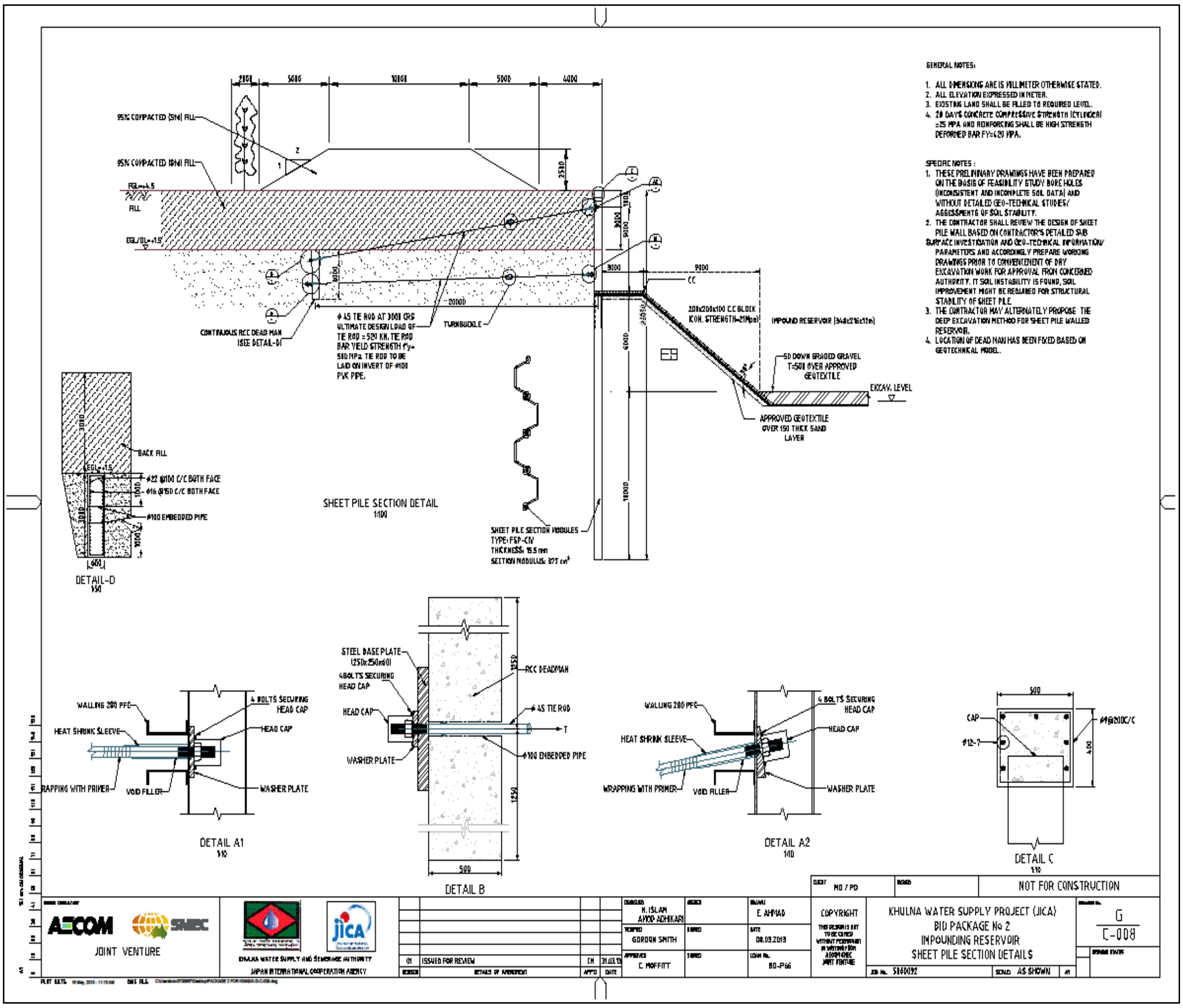

Figure 3b: Sheet pile section details

3. NUMERICAL MODELLING

The use of the sheet piles as temporary support system for the current problem of study must satisfy the limits imposed by the design specifications and also addresses the constructability of the impounding reservoir. Finite element analysis is used, which is vital not only in the evaluation of the behavior and design of the support system, but also in the evaluation of its impact on adjacent structures. The analysis is carried out using the two-dimensional FE program PLAXIS 2D 8.6 version [Brinkgreve, 2002]. PLAXIS 2D is a finite element package intended for the two dimensional analysis of deformation and stability in geotechnical engineering. It is a robust and user-friendly finite element package, developed for Geotechnical Engineering. A plane strain analysis is adopted using 15 node triangular elements as the sheet pile wall is relatively long in out of plane direction. The elasto-perfectly plastic Mohr-Coulomb Model is used to simulate the behavior of the soils in all the layers and interface between the soil and structures to simulate the contact behavior. The structural elements and solid elements for the dead end blocks are assumed to behave elastically. The first step in any FE-analysis of this geotechnical problem was to convert the data from the geotechnical reports to a simplified soil profile, idealize the structural elements and to determine the extent of the model geometry.

3.1 Geometry

The dimension of the model is 100 m wide and 50 m high. The geometry is shown in the Figure 4. The displacements are prescribed to be zero in both x- and y-direction in the bottom and only in the x-direction at the sides. The width of the model was derived by iteration, i.e. it was increased until the result was somewhat independent of the width. Three different elements are present in the model; 15-node element for the clusters and dead end block, plate element for the sheet pile wall and node to node anchor element for anchor ties, and interface element for the interaction between the soil and structural elements.

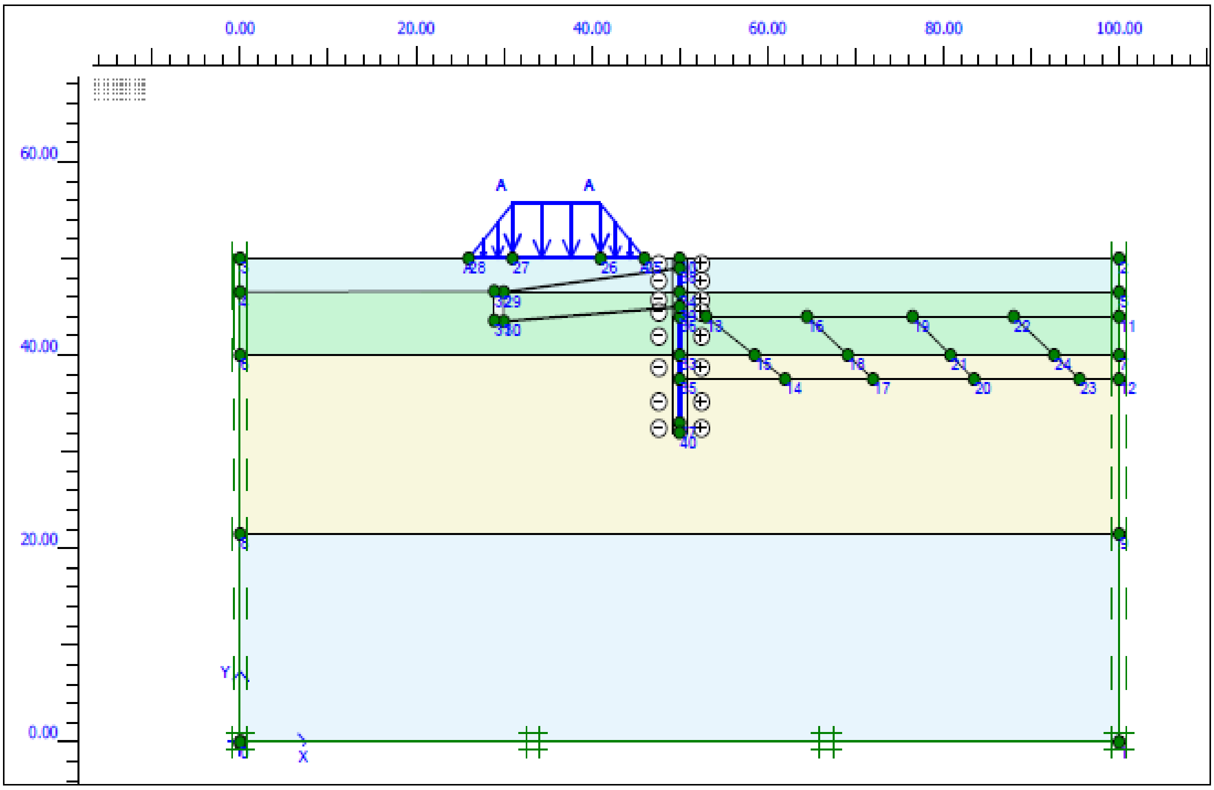

Figure 4: Geometry of the model

3.2 Mesh Generation

The mesh (Figure 5) was defined as medium dense and refined around the sheet pile wall and refined further at the bottom of the wall, as large stress gradients are expected there. Interface elements are drawn beneath the wall to smoothen the mesh, which is recommended by Plaxis in areas with high stress and strain gradient.

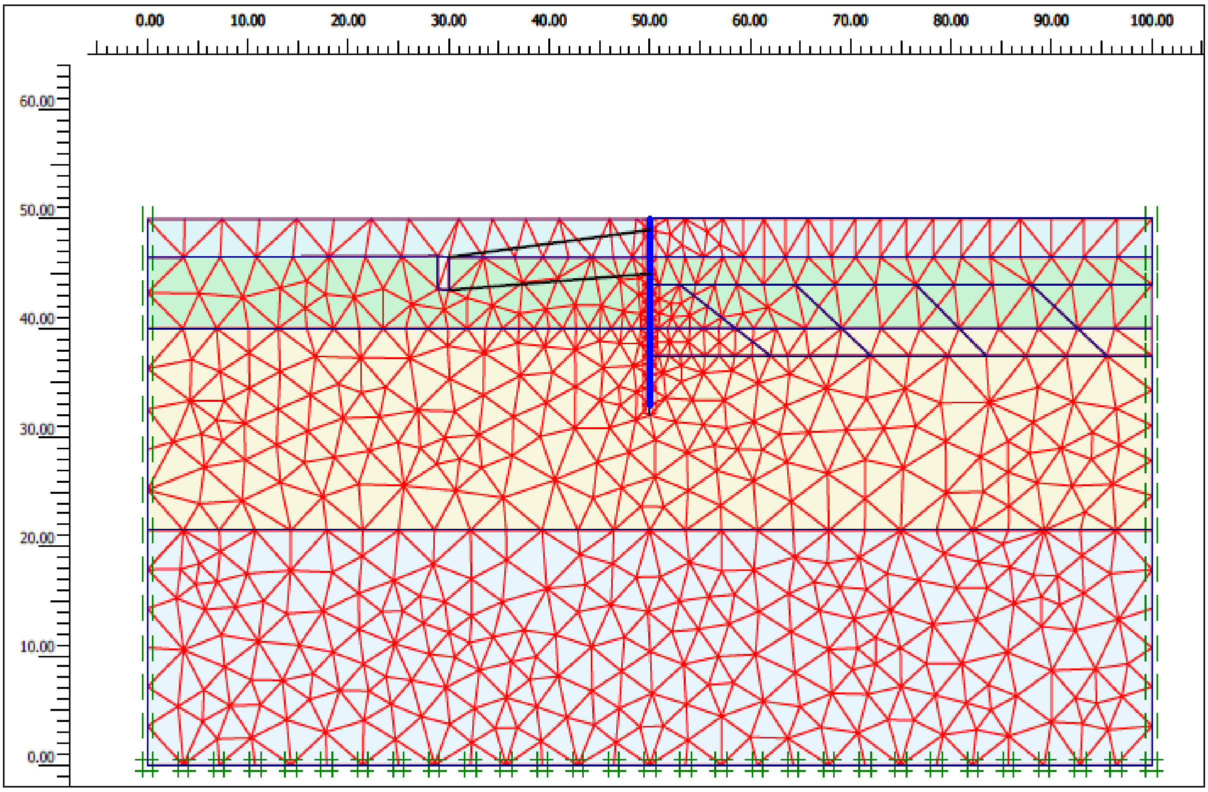

Figure 5: Discretization of the model

3.3 Material Properties

Table 1 shows the material properties of the four geotechnical layers and interfaces between the continuum soil and the structures. Table 2 shows the equivalent material properties input in the PLAXIS for the per unit length of the proposed sheet pile wall. Table 3 and 4 show the material properties used for the ties and dead end blocks.

Table 1: Material Properties for Soil Layers and Interfaces (order: top to bottom)

Parameter	Fills (Thick:3.5m)	Clayey SILTS (Thick:6.5m)	Silty SANDS (Thick:18.5m)	Deep SANDS (Thick:21.5m)	Unit
Material model	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb	-
Analysis type	Drained	Drained	Drained	Drained	-
Soil unit weight above phreatic level	16	16	16	18	kN/m^3
Soil unit weight below phreatic level	19	18	19	20	kN/m^3
Young's modulus (constant)	15000	10000	15000	20000	kN/m^2
Poisson's ratio	0.3	0.35	0.3	0.3	-
Cohesion	1	25	5	1	kN/m^2
Friction angle	28	1	30	33	degrees
Dilatancy angle	2	0	1	5	degrees
Interface strength reduction factor	0.6	0.6	0.67	0.7	-

Table 2: Material Properties for Sheet Pile Wall

Parameter	Value	Unit
Material Model	Elastic	-
Normal stiffness	$48X10^5$	kN/m
Bending stiffness	$0.25X10^5$	kNm^2/m
Poisson's ratio	0.1	-

Table 3: Material Properties for Ties (node-to-node anchor)

Parameter	Value	Unit
Material Model	Elastic	-
Normal stiffness	$3X10^5$	kN/m
Spacing out of plane	3	m

Table 4: Material Properties for Concrete Dead Block

Parameter	Value	Unit
Material Model	Elastic	-
Unit Weight	25	kN/m^3
Young's modulus	$230X10^5$	kN/m^2
Poisson's ratio	0.15	-

3.4 Initial Conditions

The initial stress-state was calculated with the K0-procedure and the initial water condition was calculated by the direct method, using the phreatic level. For this calculation no elements were activated.

3.5 Calculations

The calculation was performed as a plastic calculation and with standard settings for the iterative procedure. The calculation was performed in seven stages (Table 5) to simulate the excavation process, these are; the initial phase which correspond to the initial condition, activating the plate element to simulate the pile driving, deactivating clusters in three stages to simulate the excavation, and finally applying the fill and embankment load. For every excavation stage a steady-state ground water flow calculation was performed, (i.e. to simulate the pumping) with a water table 0.1m beneath the excavation floor, to avoid numerical problems [Moormann, 2004]. The interface elements were activated during the groundwater calculation to prevent flow through the wall.

Table 5: Calculation phases

Stage	Description
Phase 0	Initial stress calculation
Phase 1	Installation of 17m sheet pile wall, dead block and 1st tie and pre-stressing to 90 kN
Phase 2	1st Excavation to -2.5m from the EGL followed by dewatering to the next excavation level
Phase 3	Installation of 2nd tie and pre-stressing to 180 kN
Phase 4	2nd Excavation to -6.5m from the EGL followed by dewatering to the next excavation level
Phase 5	3rd Excavation to -9m from the EGL leaving a berm on the pile side
Phase 6	Placement of Fill and Embankment

4. RESULTS AND DISCUSSION

A parametric study is a study of the effect on the solution or behavior of a problem by varying the value of one parameter while keeping all other parameters at a constant or reference value. By doing so, the sensibility of the performance of the problem, in this case excavation, to each model parameter or geometry or others can be identified. The finite element method (FEM) provides the best condition for parametric studies.

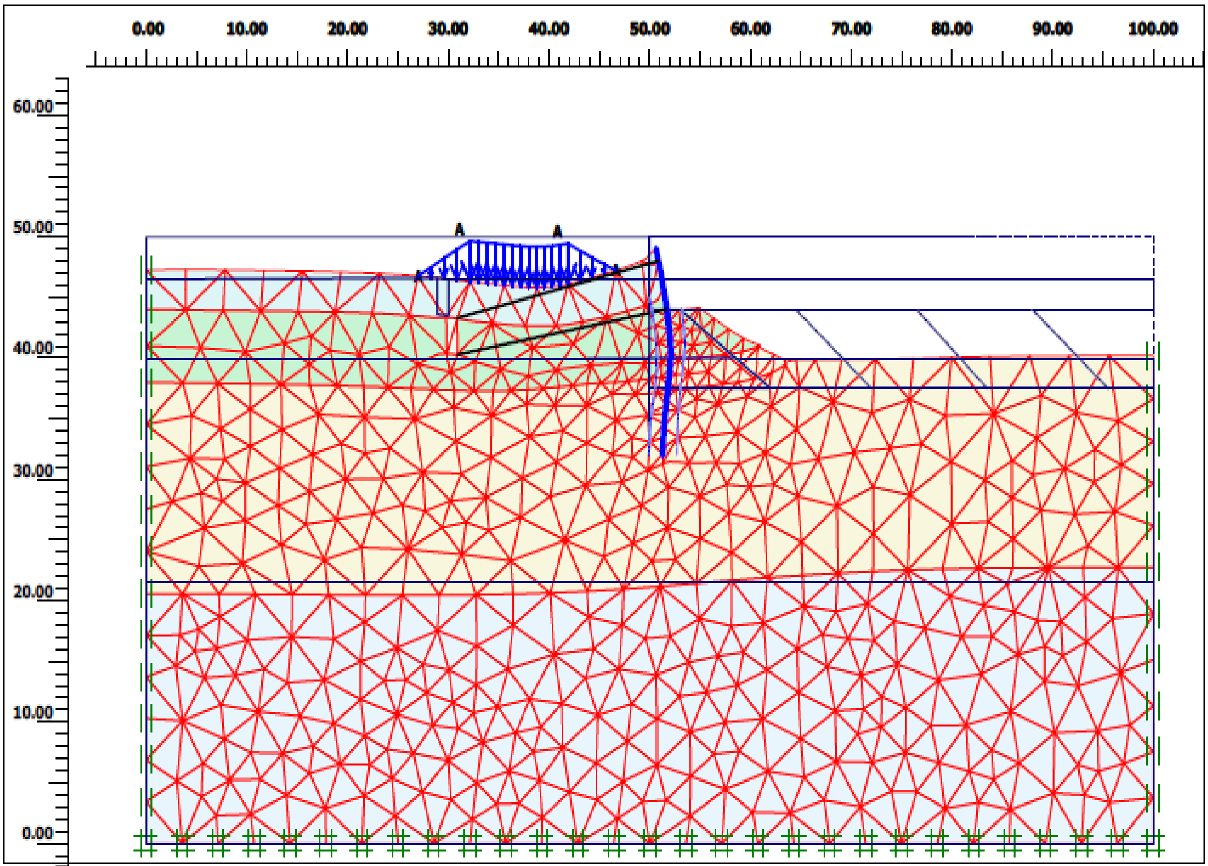

Figure 6: Deformed mesh (Max=262mm)

In this study, a detailed parametric study was conducted varying the material parameters and the support system to search the optimum design. The numerical results of the optimum design with 17m sheet pile wall are given here. Figure 6 and 7 shows the deformation pattern induced due to the excavation and embankment load as well as its interaction of sheet pile wall system. The deformation is localized below the embankment due the construction of embankment. In this study, the maximum horizontal displacement (Figure 10) along the sheet pile wall is found as 0.9% H at the excavation level, which is acceptable according to the guidelines given in [Moormann, 2004]. The Figures 8-9 show the forces induced in the sheet pile wall, which is also in allowable limits. The overall factor of safety was found 1.3 using the phi-c reduction methodology of the PLAXIS software. Berms provide support to the retaining structures before the bottom slab are installed in place. The efficiency of the berms in reducing the movement of the wall and the soil has also been investigated.

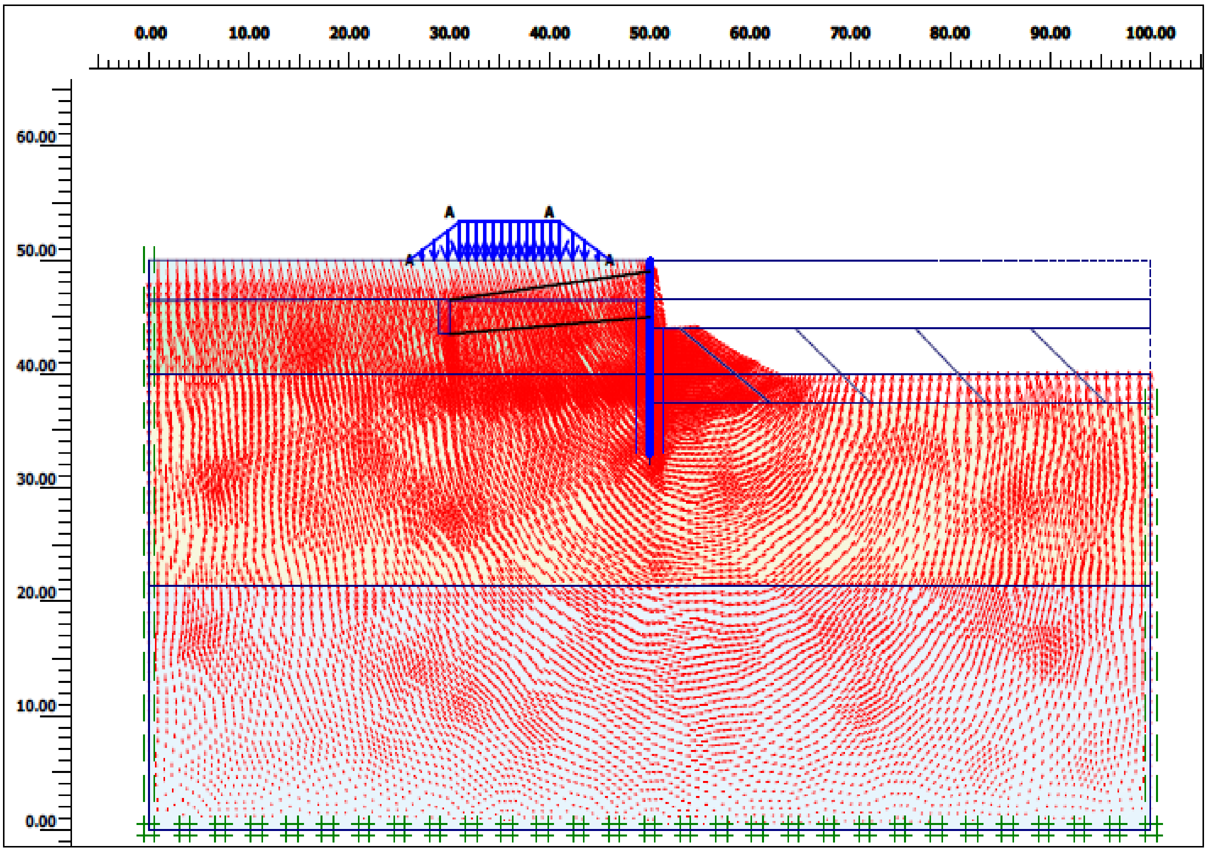

Figure 7: Total displacement vectors (Max=265 mm)

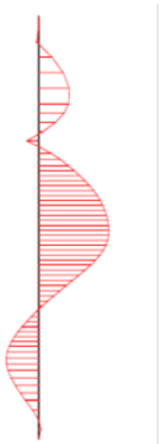

Figure 8: Bending moments in Pile (Max=93 kNm/m)

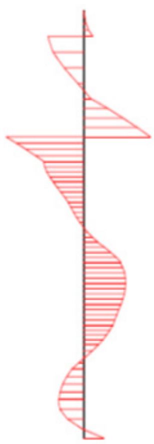

Figure 9: Shear force in (Max SF=75kN/m) and Forces in the top and bottom ties are 83 kN/m and 240 kN/m, respectively.

Figure 10: Horizontal displacement of the wall (Max=103 mm)

5. CONCLUSIONS

For the excavation of the impounding reservoir of the water treat plant for the Khulna Water Supply project of Khulna WASA, a sheet pile support system is designed using Finite Element Method. This sophisticated design procedure is able to make a direct and quantitative analysis of ground movements associated with excavation of such a complex interaction problem. From the anlysis it is observed that the ground deformations arround the exavation and stresses in the sheet pile wall and anchor system are within acceptable limit.

REFERENCES

Borja, R.I. (1990), "Analysis of Incremental Excavation Based on Critical State Theory", Journal of Geotechnical Engineering, ASCE, 116(6), 964-985.

Brinkgreve, R.B.J. (2002), "PLAXIS: Finite element code for soil and rock analyses", Balkema.

Clough, G.W. and O'Rourke, T. D., (1990), "Construction Induced Movements of in Situ Walls", In Proc. Design and Performance of Earth Retaining Structure, ASCE Special conference, Ithaca, New York, 439-470.

Burland, J.B. and Hancock, R.J.R. (1977), "Underground Car park at the House of Commons", London Geotechnical Aspects, The Structural Engineer, No. 2, Vol. 55, 87-100.

Finno, R.J., Atmatzidis, D.K. and Perkins, S.B. (1989), "Observed Performance of a Deep Excavation in Clay", J. Geotech. Engr., ASCE, Vol. 115, No.8, 1045-1064.

Goldberg, D.T., Jaworski, W.E., and Gordon, M.D. (1976), "Lateral Support Systems and Underpinning", Rep. FHWA-RD-75-128, Federal Highway Administration, Washington D.C.

Lee, F.F., Yong, K.W., Quan, K.C.N. and Chee, K.T. (1998), "Effect of Corners in Strutted Excavations: Field Monitoring and Case Histories", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 124 (4); 339-349.

Moormann, C. (2004), "Analysis of Wall And Ground Movement Due to Deep Excavation in Soft Soil Based on a New Worldwide Database", Soils and Foundations, Vol. 44, No. 1, 87-98.

O'Rourke, T.D. (1981), "Ground Movements Caused by Braced Excavations", J. Geotech. Engr. Div. Am. Soc. Civ. Engr. 107, GT9, 1159-1178.

Ou, C.Y. and Chiou, D.C. and Wu, T.S. (1996), "Three Dimensional Finite Element Analysis of Deep Excavations", Journal of Geotechnical Engineering, Vol. 122, No.5, May, 1996, 337-345.

Peck, R.B. (1969), "Deep Excavations and Tunneling in Soft Ground: State of the Art Report", Proc. of the VII ICSMFE, Mexico, 225-290.

Schweiger, H.F. and Freiseder, M. (1994), "Three Dimensional Finite Element Analysis of Diaphragm Wall Construction", Proc. of the Eighth Int. Conf. on Computer Methods and Advances in Geomechanics, 8(3), 2493-2498.

Tabrizi,K.S., Gucunski,N., and Maher, M.H. (1995), "3-D Analysis of Excavation of A Soil-Nail Wall", Second Congress on Computing in Civil Engineering, ASCE, 1, 812-819.

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STUDY ON THE MECHANICAL BEHAVIOR OF SOFT SOIL REINFORCED WITH COIR FIBER

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ABSTRACT

Many research work was carried out on the improvement of mechanical behavior of soils by the inclusion of randomly distributed natural and synthetic fibers. This paper depicts on the improvement of unconfined compressive strength (UCS) and shear strength parameters (cohesion and angle of internal friction) for artificially treated soil reinforced with coir fiber. To evaluate the influence of fiber on soil response, unconfined compression test and direct shear test for both unreinforced soil and fiber reinforced soil were performed in laboratory. In this study, the length of coir fiber was kept from 10mm to 20 mm and different selected percentage (0%, 0.25%, 0.5%, 0.75%, 1 % and 1.25%) of coir fiber were mixed with samples. Total of 63 number unconfined compressive test sample were prepared to investigate unconfined compressive strength and total 189 direct shear test samples were prepared to investigate shear strength parameters for nylon fiber reinforced soil. The test results show that fiber insertion in the soils causes an increase in UCS and shear strength parameter. The optimum fiber content for UCS, cohesion and angle of internal friction is found at 0.75% in maximum cases. Finally, it can be accomplished that UCS and cohesion is increased by 75% and 195% respectively with the comparison of unreinforced soil.

Keywords: *Coir Fiber, Synthetic Fiber, UCS, Cohesion, Angle of Internal friction*

1. INTRODUCTION

Now a day, due to urbanization population is increasing day by day but the amount of land cannot cope with the construction of structure on it. On the other hand, many lands are available now which possess soft soil like clay soil. It has got much possibilities of the structure to be failed if constructed on soft soil because this type of soil is less stable. Generally, clay soil has low shear strength, low tensile strength and high compressibility. As a result, it may cause excessive settlement of the structure constructed on it. In order to use the land having soft soil for construction purposes, improvement technique must be applied to the soil. There are several soil improvement method have been applied including soil replacement, dynamic compaction, lime/ cement column, stone column and soil reinforcements with fibrous materials. Using fiber reinforcement in soft soil is one of the most effective improvement techniques.

Fiber reinforcement improves shear strength and tensile strength of the soft soil. It is generally of two types like natural fiber and artificial fiber. Coir, jute, cotton, papyrus, bamboo, palm etc. are usually used as natural fiber whereas artificial fiber comprises Nylon, Plastic, polypropylene fiber etc. Reinforcing soil with natural fiber is an old practice to improve the properties of soil like bearing capacity, shear strength (c and ϕ). In fifth and fourth millennium B.C, Natural fibers were used to reinforce on soil for construction purpose (Jones, C.J 1985). Vidal (1969) first showed that, inserting fiber as reinforcing element in soil increases shear resistance of soil. Natural fiber material is very cheap and easily available in local area. It is biodegradable and environment friendly. Many recent studies have been performed on fiber reinforcement of soft soil. In these studies soil is mixed with papyrus fiber in percentages of 5%, 10%, 15%, and 20% of the volume of raw soil. But maximum shear strength was obtained in a percentage of 10%. (Aqeel Al Adili, Rafig Azzam, Giovanni Spagnoli, and Joerg Schrader, 2011). Coir fiber also improves geotechnical properties and mechanical behavior of soft clay soil (Sivakumar Babu J.L. et al. 2008).

Recently, some artificial fiber is also inserted in soil such as Nylon, polyester fiber. Inserting artificial fiber also improved the engineering properties of soil. An investigation shows that polyester fiber was mixed with clay in five different percentages of 0%, 0.2%, 0.4%, 0.6%, 0.8, and 1.0% of the soil mass where 1.2 cm fiber length

was used. Maximum shear strength was obtained in a percentage of 10% of the soil mass (Kalpana Vinesh Maheshwari, Atul K. Desai, Chandresh H. Solanki, 2011). Another investigation was shown that clay mixes nylon fiber in percentages of 10%, 20%, 30% of the soil mass and fiber length was 4mm. But here shows maximum strength of clay in unreinforced situation (A. R. Estabragh, A. T. Bordbar, A. A. Javadi, 2011). From above two investigation, we see that Artificial fiber is need to mix with soil with low percent of soil mass to get better engineering properties of soil. In modern age, geo-textiles, geo-grides are also used to reinforce soil.
Fiber reinforcement is randomly distributed while mixing with soil. Some geotechnical or soil mechanics related tests need to conduct to measure strength and bearing capacity of soil. The tests, which were conducted in maximum investigations, are unconfined compression test, CBR test, direct shear test, triaxial test and compaction test. (Michalowski and Cermak, 2003; Kaniraj and Havangi, 2001; Kaniraj and Gayatri, 2003; Gosavi et al., 2004, Yetimoglu et al., 2005; Andersland and Khattak, 1979: Hoare 1979: Gray and Ohashi, 1983; Maher and Gray, 1990; Charan, 1985; Michalowski and Zhao, 1960).There is less number of investigations where Model footing test was conducted because it needs large sample. Comparison between natural and artificial fiber, Less amount of percentage of artificial fiber are need mix with soil, where natural fiber needs to mix more percentage of soil mass to get better engineering properties of soil.(Gupta, 2008).

This experimental program is conducted with soft clay soil (below 5ft from ground surface), which was mixed with different percentages of coconut coir fiber of the soil mass as reinforcing element. Fibers were randomly distributed with soil. Unconfined compression test and direct shear test were conducted for the fiber reinforced soil.

2. EXPERIMENTAL STUDY

2.1 Collection and specification of soil

The grayeish clay soil was collected from Khulna region. Collected soil was very wet. For removing moisture of soil, it was dried 24 hours in oven at 105ºC. Then the soils were screened through No.4 sieve (4.75mm). For determination of geotechnical properties of soil, Standard proctor compaction test, Atterberg limit test, specific gravity test was conducted in laboratory. The soil was classified as CH (Clay with High Plasticity) according to Unified Soil Classification systems (USCS). The Geotechnical properties of soil are shown at Table 1.

Table 1: Geotechnical properties of soil

Soil properties	Value
Liquid Limit	50.35%
Plastic Limit	26.06%
Plasticity Index	24.29%
Specific Gravity	2.56
USCS classification	CH
Optimum water content	22.4%
Maximum dry density	26.2 Kg/m^3

2.2 Specification of Fiber

In the present study, out of the different natural fiber, coconut coir fiber was selected for performing the test associated with the soil strength. Coconut fiber is selected as it is easily available to the locality and slowly biodegradable comparing to the others natural fiber. It has also good adhesion with soil particle and good potential to absorb energy. Fiber samples were made from coconut coir rope, which was collected from market. Then it was resized and made fiber sample. The length and diameter of fiber samples were kept approximately from 10-25mm and 0.1 mm, respectively as shown in Figure 1. The specification for the selected fiber is shown in Table 2, which depicts the strength and elastic modulus is 176 MPa and 20 MPa, respectively.

Figure 1: Collection of fiber sample

The specification for the selected fiber is shown in Table 2, which depicts the strength and elastic modulus is 176 MPa and 20 MPa, respectively.

Table 2: Properties of coir fiber

Fiber properties	**Value**
Length	10-25mm
Diameter	0.1mm
Color	Brown
Cross section type	Circular
Tensile strength	176MPa
Young's modulus	5GPa
Elastic modulus	20MPa

2.3 Sample preparation

Fiber sample having average length from 10-25mm was taken for this investigation. This Fiber samples were randomly mixes with soil with different percentage ranging from 0% to 1.25% with the increment of 0.25% with respect to dry weight of soil. It was mixed thoroughly with four to five minutes in a tray. Then optimum water content was added into soil and fiber sample, which was taken from standard proctor compaction test of unreinforced soil. Then it was mixed thoroughly about four to five minutes with great care for obtaining a uniform mixture. The specimen was prepared with the help cylindrical split mold that was used in the standard proctor test.

2.4 Test conducted

For determination of strength behavior or mechanical behavior of soil randomly reinforced with different percentage of fiber, a series of unconfined compression test and direct shear test were conducted in laboratory. For determination and effect of unconfined compression strength of fiber reinforced sample, unconfined compression test was conducted in laboratory. For determination and effect of shear strength parameters (Cohesion and angle of internal friction), Direct Shear test was conducted in laboratory. There were total 63 number of unconfined compression test and same number of direct shear test was conducted in laboratory for fiber reinforced sample. Fiber samples were randomly mixes with soil with different percentage ranging from 0% to 1.25% with the increment of 0.25% with respect to dry weight of soil.
Unconfined compressive test have been performed on unreinforced and reinforced sample, for finding out the effect of fiber reinforcement of unconfined compressive strength of soft clay soil. A series of unconfined compression test were conducted of fiber reinforced sample in laboratory according to ASTM D-2166. A

cylindrical specimen was made for unconfined compression test and compacted with three layers with adding of optimum water content of soil. The height of cylindrical specimen was 2.8 inch and the diameter of cylindrical specimen was 1.4 inch (36mm diameter and 72mm length). Cylindrical specimens were prepared at 0%, 0.25%, 0.5%, 0.75%, 1% and 1.25% of fiber content. Total three samples were made for per percentage of fiber content The characteristics of stress- strain of unreinforced and reinforced sample were determined by unconfined compression testing machine.

A series of Direct Shear test were conducted of fiber reinforced sample in laboratory according to ASTM D-3080. A 60mm x 60mm square specimen was made for conducting direct shear test with adding of optimum water content of soil and the height of specimen was 25mm. Shear test specimen were prepared in a percentage of 0%, 0.25%, 0.5%, 0.75%, 1% and 1.25%. The horizontal shear rate or motor speed of Direct Shear test machine was kept at 5mm/min. Direct shear test was conducted for normal stress, σ=27.25, 54.5 and 81.75KPa. The value of shear stress were recorded as a function of horizontal deformation up to maximum value allowed by direct shear test machine.

3. RESULT AND DISCUSSION

3.1 Effect of unconfined compressive strength

The value of unconfined compressive strength is shown in Table 3 and Table 4 for all length of fiber. Unconfined compressive strength of reinforced soil was increased with the increasing percentage of fiber. But it was decreased with the increasing of fiber length. Maximum unconfined compressive strength was found at 0.75% fiber in 10 mm length. The unconfined compression strength was 75% higher over plain soil. This increment of unconfined compressive strength of soil is due to bridge effect of the fiber, which can effect to impede further development of failure plain and deformation of soil (Zaimoglu and Yetimoglu, 2011).

Table 3: Unconfined compressive strength of coir fiber reinforced soil for 10mm and 15mm

% of Fiber	Unconfined compressive strength (kPa)							
	10 mm				15mm			
	1st sample	2nd sample	3rd sample	Average value	1st sample	2nd sample	3rd sample	Average value
0	63.63	61.86	62.75	62.75	63.63	61.86	62.75	62.75
0.25	87.65	84.78	86.23	86.22	85.39	85.13	85.5	85.34
0.5	99.09	97.81	98.57	98.49	93.28	95.01	94.6	94.3
0.75	111.38	111.09	111.56	111.34	102.93	100.23	102.3	101.8
1	110.52	107.22	109.2	108.98	99.86	97.88	98.9	98.88
1.25	103.33	102.44	102.9	102.89	97.23	95.67	96.5	96.47

Table 4: Unconfined compressive strength of coir fiber reinforced soil for 20mm and 25mm

% of Fiber	Unconfined compressive strength (kPa)							
	20 mm				25mm			
	1st sample	2nd sample	3rd sample	Average value	1st sample	2nd sample	3rd sample	Averag e value
0	63.63	61.86	62.75	62.75	63.63	61.86	62.75	62.75
0.25	73.69	79.54	77.2	76.8	66.5	71.72	69.3	69.2
0.5	86.5	90.09	88.2	88.3	74.92	82.55	69.2	75.6
0.75	98.39	96.74	98.3	97.8	83.84	87.11	84.9	85.3
1	94.28	92.37	93.3	93.3	78.89	75.64	77.2	77.3
1.25	87.85	85.59	87.3	86.9	74.43	71.17	72.3	72.6

Variation of unconfined compressive strength with fiber content for reinforced (10mm, 15mm, 20mm & 25mm) soil specimen is presented in Figure. 2. The unconfined compressive strength of specimens is found to increase with the fiber content. However, the rate of increase of strength with fiber content is not linear. Initially the rate of increase is high thereafter the same is not that much prominent. Randomly oriented discrete inclusions incorporated into granular materials improve its load – deformation behavior by interacting with the soil particles mechanically through surface friction and also by interlocking. The bonding and interlocking between the granular particle and reinforcement facilitates the transfer of the tensile strain developed in the mass to the reinforcement and thus, the tensile strength of the reinforcement is mobilized and helps in improving the load capacity of the reinforced mass. The test result shows that the failure stress of reinforced specimen's increases with fiber content for all fiber length. The plots reveal that at given compacted density and fiber content, the 10mm size fiber gives higher strength than other size of fibers. The fibers modifies the stress condition in the specimens and transfer the shear along the failure plane to the surrounding mass by combined effect of adhesion and friction between the fiber and soil particles. For other size of fibers sufficient anchorage to fiber might not be developed leading to pull-out failure. The plots also reveal that 0.75% of fiber content gives maximum strength for all size of fibers. In the present case only four fiber lengths have been tried. However it is expected that for given compacted density an optimum fiber length and fiber content can be arrive at which mobilizes the optimum strength of the fiber.

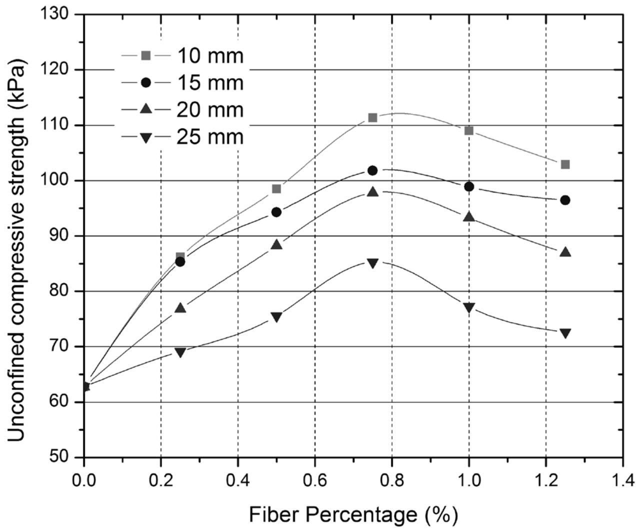

Figure 2: Variation of Unconfined Compressive strength at different percentage of fiber

3.2 Effect of shear strength parameters

The value of Shear strength parameter is shown in tabular form in Table 5, 6, 7 and 8. Shear strength parameters of reinforced soil was increased with respect to increasing percentage of fiber. Maximum value of cohesion was found at 0.75% fiber in 10 mm length and maximum value of angle of internal friction was found at 0.75% fiber in 10 mm length. The cohesion strength was 195% higher over plain soil.

Table 5: Cohesion strength of coir fiber reinforced soil for 10mm and 15mm

Cohesion (kPa)								
% of Fiber	10 mm				15mm			
	1st sample	2nd sample	3rd sample	Average value	1st sample	2nd sample	3rd sample	Average value
0	16.91	17.23	17.07	17.07	16.91	17.23	17.07	17.07
0.25	30.38	32.44	31.52	31.447	28.24	27.98	28.35	28.19
0.5	34.96	35.77	35.47	35.4	33.44	31.76	32.6	32.6
0.75	50.04	47.17	48.52	48.577	47.55	45.88	46.58	46.67
1	45.78	44.15	45.12	45.017	43.17	41.32	42.58	42.3567
1.25	40.61	41.72	41.165	41.165	39.2	38.92	39.12	39.08

Table 6: Cohesion strength of coir fiber reinforced soil for 20mm and 25mm

Cohesion (kPa)								
% of Fiber	20 mm				25mm			
	1st sample	2nd sample	3rd sample	Average value	1st sample	2nd sample	3rd sample	Average value
0	16.91	17.23	17.07	17.07	16.91	17.23	17.07	17.07
0.25	26.48	27.31	26.8	26.863	24.09	23.11	23.7	23.63
0.5	30.99	29.73	30.42	30.38	30.31	27.39	28.6	28.77
0.75	46.39	44.76	45.28	45.477	39.07	37.67	38.6	38.45
1	31.55	37.13	34.42	34.367	35.48	33.16	34.2	34.28
1.25	30.93	33.79	32.41	32.377	33.66	30.42	32.5	32.19

Table 7: Angle of internal friction of coir fiber reinforced soil for 10mm and 15mm

Angle of internal friction (Degree)								
% of Fiber	10 mm				15mm			
	1st sample	2nd sample	3rd sample	Average value	1st sample	2nd sample	3rd sample	Average value
0	23.06	23.02	23	23.04	23.06	23.02	23	23.04
0.25	24.65	25.03	15.2	21.627	23.98	24.34	24.3	24.2
0.5	27.02	27.56	27.3	27.293	26.42	27.05	26.5	26.7
0.75	27.56	27.91	28.5	27.99	27.11	28.38	27.8	27.8
1	26.83	26.72	27.3	26.95	26.42	25.95	26.3	26.2
1.25	26.51	26.37	26.6	26.493	26.01	25.13	25.8	25.6

Table 8: Angle of internal friction of coir fiber reinforced soil for 20mm and 25mm

% of Fiber	Angle of internal friction (Degree)							
	20 mm				25mm			
	1st sample	2nd sample	3rd sample	Average value	1st sample	2nd sample	3rd sample	Average value
0	23.06	23.02	23	23.04	23.06	23.02	23.04	23.04
0.25	21.92	22.63	22.5	22.3333	21.6	21.98	21.82	21.8
0.5	24.72	25.07	24.8	24.8633	23.07	24.16	23.65	23.627
0.75	25.84	26.21	26.3	26.1167	24.37	24.76	24.62	24.583
1	29.72	24.56	27.2	27.16	24.18	24.11	24.25	24.18
1.25	27.3	24.12	25.3	25.5867	23.41	22.92	23.18	23.17

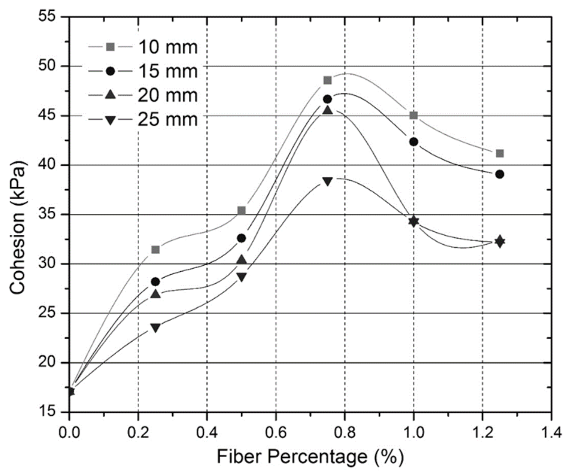

Figure 3: Variation of Cohesion strength at different percentage of fiber

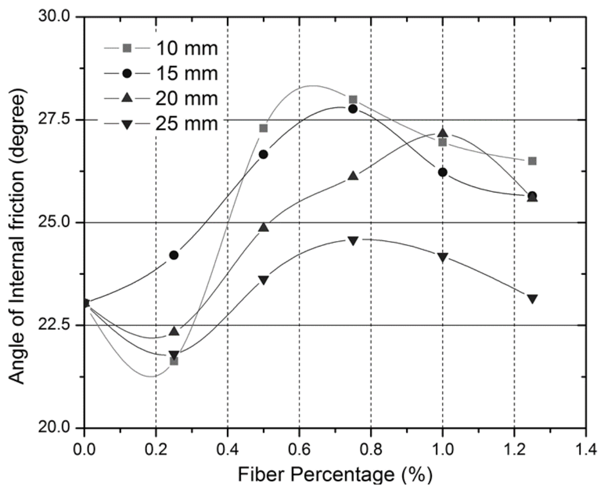

Figure 4: Variation of Angle of internal friction at different percentage of fiber

Figure. 3 And Figure.4 shows the variation of cohesion and angle of internal friction with fiber content for reinforced (10mm, 15mm, 20mm & 25mm fiber) soil specimens compacted at standard density. The cohesion of reinforced specimens is found to increase with the fiber content. However, the rate of increase of cohesion with fiber content is not linear. Initially the rate of increase is high thereafter the increase in cohesion is not that prominent. Similar trend is also observed between the angles of internal friction with fiber content. The plots reveal that the 10mm size fiber is more effective than other size fibers in maximum case. The fibers modifies the stress condition in the specimens and transfer the tensile strain along the failure plane to the surrounding mass by combined effect of adhesion and friction between the fiber and soil particles. For other length of fibers, sufficient anchorage to fiber might not be developed leading to pull-out failure and lesser mobilization of fiber capacity. The plots also reveal that the 0.75% of fiber content is optimum fiber content for all size of fibers in maximum cases. In the present case only four fiber lengths have been tried. However it is expected that for maximum dry density an optimum fiber length and fiber content can be arrive at, which mobilizes the optimum strength of the fiber.

4. CONCLUSION

For the completion of this study, a series of unconfined compression test and direct shear test were conducted in laboratory on fiber reinforced sample. Coconut coir fiber were used as fiber sample. The optimum fiber content for unconfined compression strength is 0.75%. Unconfined compressive strength was significantly increase with the comparison of unreinforced soil, which is around 55%. From the test result of direct shear test, maximum cohesion of fiber reinforced soil was 48.577kPa at 0.75% of fiber at 10mm length. Cohesion was increased around 195% with comparison of unreinforced sample. Angle of internal friction at 0.75% of fiber at 10mm fiber length and does not have significant increment. Fiber reinforced sample was more ductile than unreinforced sample.

REFERENCES

ASTM D 2166 (2000) Standard test method for unconfined compressive strength of cohesive soil. American Society for Testing and Materials, West Conshohocken.

ASTM D 3080 (2004) Standard test method for direct shear test of soils under consolidated drained conditions. American Society for Testing and Materials, West Conshohocken.

Maher MH, Ho YC (1994) Mechanical-properties of kaolinite fiber soil composite. J Geotechnical Engineering 120(8):1381–1393.

Tang CS, Shi B, Gao W, Chen F, Cai Y (2007) Strength and mechanical behavior of short polypropylene fiber reinforced and cement stabilized clayey soil. GeotextGeomembr25 (3):194–202.

A SahinZaimoglu, TemelYetimoglu (2011) Strength Behavior of Fine Grained Soil Reinforced with Randomly Distributed Polypropylene Fibers. GeotechGeolEng (2012) 30:197–203.

Jones, C.J. 1985, Earth reinforcement and soil structure. Butterworth’s advanced series in geotechnical engineering.

A. R. Estabragh, A. T. Bordbar, A. A. Javadi, Mechanical Behavior of a Clay Soil Reinforced with Nylon Fibers. GeotechGeolEng (2011) 29:899–908.

Aqeel Al Adili, RafigAzzam, Giovanni Spagnoli ,Joerg Schrader, STRENGTH OF SOIL REINFORCED WITHFIBER MATERIALS (PAPYRUS).Soil Mechanics and Foundation Engineering, Vol. 48, No. 6, January, 2012 (Russian Original No. 6, November-December, 2011).

OmidAzadegan, E. AkbariKaffash, M.J. Yaghoubi, G.R Pourebrahim, Laboratory Study on the Swelling, Cracking and Mechanical Characteristics of the Palm Fiber Reinforced Clay.

Kalpana Vinesh Maheshwari, Atul K. Desai, Chandresh H. Solanki, Performance of Fiber Reinforced Clayey Soil.

Vidal, H. (1969). "The Principle of Reinforced Earth," Highway Research Record, No. 282, NCR-HRB, Washington, D.C., pp. 1-16.

Gupta, P. and Saran, S. (2008) “Behavior of Fiber Reinforced Sand in Different Test Conditions” JI. Indian Geotechnical Journal, pp.272-282.

Michalowski, R. L. and Zaho, H. (1996) “Failure of Fiber – reinforced Granular Soils”,JI. Geotech. Engg. ASCE, 122 (3), pp. 226-333.

Michalowski, R. L. and Cermak, J. (2003) “Triaxial Compression of Sand Reinforced with Fibers”, J. Geotechnical and Geoenvironmental Engg, ASCE, 129 (2), pp. 125-136.

Maher, M. H. and Gray, D. H. (1990) “Static Response Sands Reinforced with Randomly Distributed Fibers”, JI. Geotech. Engg. ASCE, 116 (11), pp. 1661- 1677.

Kaniraj, S. R. and Havanagi, V. G. (2001) “Geotechnical Behavior of Fly Ash – soil Mixtures”, JI of Geotextiles and Geomembranes, 21, pp. 123-149.

Hoare, D. J. (1979) “Laboratory Study of Granular Soils Reinforced with Randomly Oriented Discrete Fibers”, Proc. Int. Conf. on Use of Fabrics in Geo-technical., Paris, France.1, pp. 47-52.Mallela, J., Quintus, H. V., and Smith, K. (2004). Consideration of lime-stabilized layers in mechanistic-empirical pavement design. The National Lime Association.

Gray, D.H. and Oshashi, H. (1983) “Mechanics of Fiber-reinforcement in Sand”, JI.Geotech. Eng., ASCE, 109 (3), pp. 335-353.

Charan, H. D. (1995) “Probabilistic Analysis of Randomly Distributed Fiber Reinforced Soil, PhD Thesis, Dept. of CivilEngg., I.I.T. Roorkee, India.

Andersland, O. B. and Khattak, A. S. (1979) “Shear Strength of Kaolinite / Fiber Soil Mixtures”, Proc. Int. Conf. on Soil Reinforcement, Paris, France. 1, pp. 11-76.

SivakumarBabu J.L. et al. (2008), “Use of Coir Fibers For Improving the Engineering Properties of Expansive Soils”, Journal of Natural Fibers, Vol. 5 (1), pp 61-75.

Rabindra Kumar Kar, Pradip Kumar Pradhan, Ashutosh Naik “Plate Load Test on Fiber- Reinforced Cohesive Soil.

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PILE CAPACITY IN KHULNA SUB-SOIL

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ABSTRACT

This paper presents the results of pile capacity in tabular and graphical form from a field and laboratory investigation of sub-soil in Khulna region. The sub-soil investigation reports of different sites of Khulna were collected from CRTS (Consultancy Research & Testing Services), Department of Civil Engineering, KUET and pile capacities at different depths were calculated. The analysis was made by using allowable skin friction and allowable end bearing capacity to determine pile capacity .A comparative study is made between pile capacities obtained by the Conventional method and pile load test. Variation of pile capacity with respect to length of the pile obtained by Conventional method differs from those obtained by pile load test. This difference can be considered reasonable from practical point of view. The results obtained from this study can be used for the design of pile in those cases where the owners are going to use pile foundation for small project without any sub-soil investigation. Moreover, for large projects pile capacity obtained by sub-soil investigation reports can be utilized for design with confidence by comparing with these results where pile load test is not performed.

Keywords: *Pile Capacity, Khulna Region, Conventional Method, Pile Load Test*

5. INTRODUCTION

Khulna City is situated on southern region of Bangladesh. The soil in this region is formed by the alluvial deposits from different rivers (Rupsha, Vairab etc.). This region is also covered by deep forest of Sundarban. Due to tectonic forces at different times in the past, these deep forests were buried underneath. For these reasons, the soil is very soft, compressible having organic matter with low bearing capacity as well as pile capacity(Molla and Malik,1997).

Khulna City is expanding with large development projects including construction of high rise buildings, bridges and overhead water tanks etc. Since the bearing capacity of the sub- soil of Khulna is very low, pile foundation is used for the construction of high rise buildings and other important structures. For most of the projects other than a large project, the owner does not like to bear the cost of pile load test .In such cases the pile capacity obtained by conventional method can be used for the design of pile foundation confidently if any published paper is available on pile capacity of Khulna sub-soil. Unfortunately, such published paper is not available till now. Therefore, it is necessary to publish such a paper.

It is well known that the sub-soil of Khulna is of critical nature. The existence of organic layer is a very disadvantageous aspect of Khulna sub-soil. This organic layer is situated almost everywhere in Khulna City. In some places the thickness of this layer is 5ft to 20ft and in some places the thickness of this layer is much more. At south of Khulna the location of this organic layer is nearer to ground surface. At most of the places the top layer is composed of clayey silt or silty clay. The thickness of this clay and silt layer including organic layer is 35 to 40ft from ground level. Somewhere sand pockets are available within these 40ft.In most places the water table varies from 2-4ft.All the above mentioned factors have effects on the pile capacity in Khulna sub-soil. The main objectives of this study were to determine the pile capacity of Khulna sub-soil by Conventional method and to compare with the values obtained by pile load test.

6. METHODOLOGY

In this study pile capacity is determined at fourteen different sites which are shown in figure-1. Details of the evaluation procedure are provided in the following sub-heading.

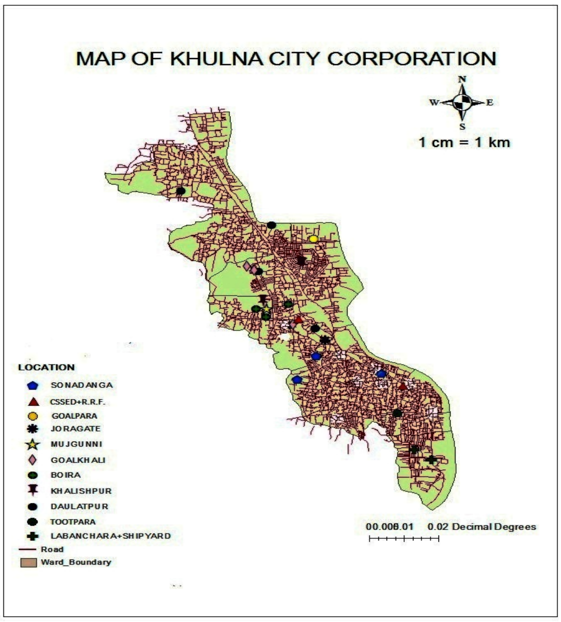


Figure-1: Location map of Study Area in Khulna Region

6.1 For Cohesive Soil

For soil under cohesive group i.e. for clay and plastic silt, the skin friction and end bearing capacities of pile may be evaluated by the following general formula:

$f_{su} = \alpha C_u$

$q_{pu} = N_c C_u$

Where,

f_{su} =Ultimate skin friction

α = Adhesion factor

C_u= Undrained shear strength

q_{pu} =Ultimate end bearing of the pile

N_c = Bearing capacity factor for deep foundation (generally 9)

6.2 For Non-cohesive Soil

For non–cohesive soil of silt, firm to medium sand, the skin friction and the end bearing capacities of pile may be evaluated by the following formula:

$f_{su} = 4N/20$

$q_{pu} = 4N$

Where,

f_{su} = Ultimate skin friction

q_{pu} = Ultimate end bearing of the pile

N = SPT value

6.3 Pile Capacity By Conventional Method

Ultimate pile capacity = $A_s*f_{su}+ A_b*q_{pu}$

Where,
A_s= Cross-sectional area of the surface of the pile = πDL (ft2)
f_{su} = Ultimate skin friction, tsf
A_b = Cross-sectional area of the base of the pile =$\pi D2$ (ft2)
q_{pu} = Ultimate end bearing of the pile, tsf
D = Diameter of the pile
L= Length of the pile

(Bowels, 1997; Singh & Chowdhury, 1994; and Teng, 2012)

7. RESULTS AND DISCUSSION

For each site pile capacity at different boreholes are determined and by averaging the pile capacities of all the boreholes the average pile capacity of that site is obtained. Finally averaging the pile capacities of all the sites the average pile capacity in Khulna sub-soil is obtained. Detail procedures of evaluation of two sites are presented in the following tables.

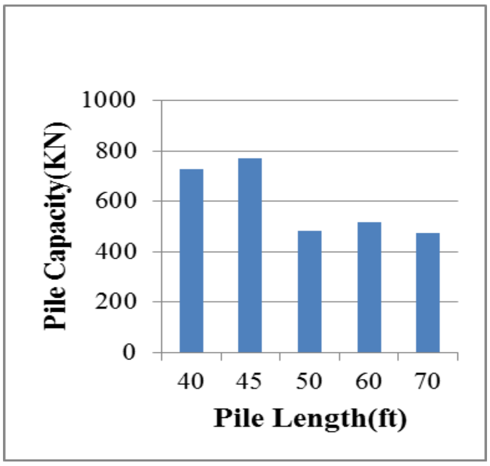
Site-1: An Infrastructure under Women Oppression Protection Project at Five Divisional Towns, Khulna

Table-1: Ultimate Pile Capacity of Pre-cast Pile by Conventional Method

Bore Hole(BH)	Length of the Pile(ft)	Ultimate Pile Capacity by Conventional Method(KN)		
		Dia(12")	Dia(18")	Dia(24")
BH-1	40	692.67	1457.95	2502.53
	45	756.63	1581.75	2704.75
	50	338.79	703.63	1198.76
	60	506.37	1038.78	1757.36
	70	475.93	965.20	1622.18
BH-2	40	646.58	1360.96	2336.07
	45	756.63	1581.75	2704.75
	50	338.79	703.63	1198.76
	60	354.43	727.09	1230.03
	70	475.93	965.23	1622.18
BH-3	40	646.58	1360.96	2336.07
	45	804.11	1681.03	2874.56
	50	484.02	1005.27	1712.67
	60	557.20	1143.11	1933.88
	70	528.72	1027.30	1802.05
BH-4	40	923.65	1944.14	3337.08
	45	756.63	1581.75	2704.75
	50	774.50	1608.56	2740.50
	60	658.31	1350.49	2284.68
	70	423.14	858.22	1442.31

Table-2: Average Ultimate Pile Capacity of Pre-cast Pile by Conventional Method

Length of the Pile(ft)	Average Ultimate Pile Capacity by Conventional Method(KN)		
	Dia(12")	Dia(18")	Dia(24")
40	727.37	1531.02	2627.94
45	768.50	1606.57	2747.45
50	484.03	1005.32	1712.67
60	519.07	1064.87	1801.49
70	475.93	953.98	1622.18



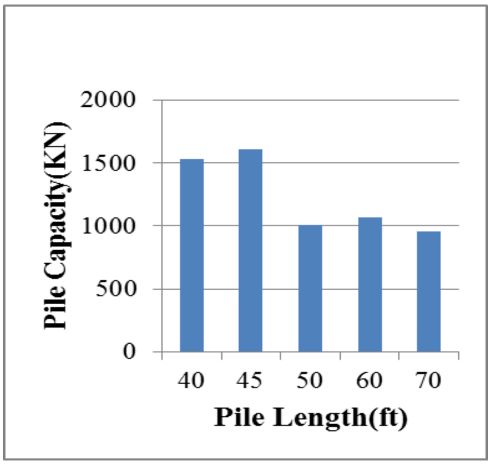


Figure-2: Variation of pile capacity for site-1 of 12" diameter pile (Left) and Variation of pile capacity for site-1 of 18" diameter pile (Right)

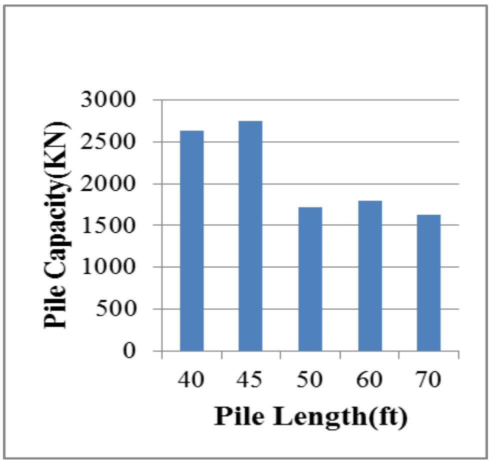


Figure-3: Variation of pile capacity for site-1 of 24" diameter pile

From the figure it is observed that pile capacities at 40ft and 45ft are greater than pile capacities at 50ft, 60ft and 70ft. Because at 40ft and 45ft the values of end bearing capacities are greater than the values at 50ft, 60ft and 70ft.

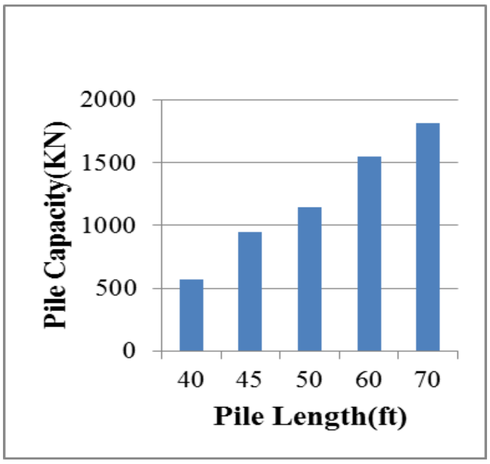
Site-2: Fabrication Shed at Khulna Shipyard, Khulna

Table-3: Average Ultimate Pile Capacity of Pre-cast Pile by Conventional Method

Length of the Pile(ft)	Average Ultimate Pile Capacity by Conventional Method(KN)		
	Dia(12")	Dia(18")	Dia(24")
40	168.31	274.45	395.27
45	240.48	391.38	562.74
50	295.56	477.94	690.88
60	566.09	1129.90	1880.92
70	740.31	1501.48	2523.31

Table-4: Ultimate Pile Capacity of Pre-cast Pile by Conventional Method

Bore Hole(BH)	Length of the Pile(ft)	Ultimate Pile Capacity by Conventional Method(KN)		
		Dia(12")	Dia(18")	Dia(24")
BH-1	40	128.20	208.0	298.30
	45	232.66	376.43	538.50
	50	254.44	409.11	582.06
	60	658.31	1350.49	2284.67
	70	634.57	1287.02	2162.90
BH-2	40	128.20	208.0	298.30
	45	205.0	331.18	473.14
	50	275.11	444.09	634.02
	60	658.31	1350.49	2284.67
	70	846.0	1715.81	2883.50
BH-3	40	228.47	374.12	540.73
	45	174.56	281.53	401.63
	50	298.57	483.26	691.55
	60	709.14	1454.81	2461.20
	70	846.0	1715.81	2883.50
BH-4	40	128.20	208.0	298.30
	45	273.44	445.55	641.27
	50	324.83	526.62	754.67
	60	348.85	558.67	792.09
	70	740.43	1501.72	2523.75
BH-5	40	228.47	374.12	540.73
	45	316.73	522.22	759.14
	50	324.83	526.62	754.67
	60	455.82	935.09	1581.95
	70	634.57	1287.01	2162.90



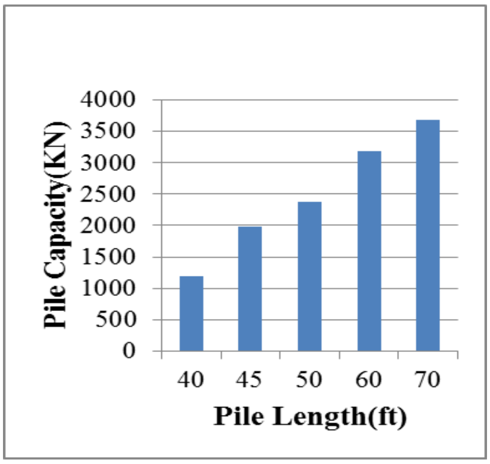


Figure-4: Variation of pile capacity for site-2 of 12” diameter pile (Left) and Variation of pile capacity for site-2 of 18” diameter pile(Right)

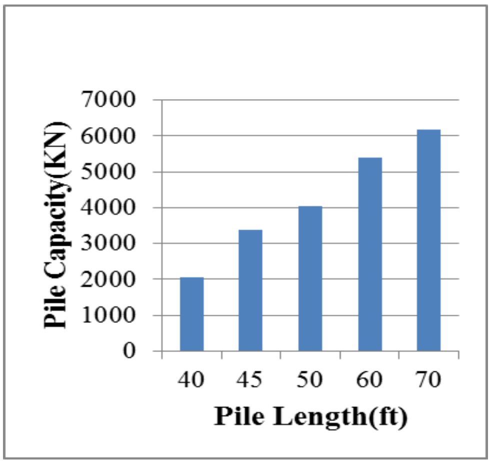


Figure-5: Variation of pile capacity for site-2 of 24" diameter pile

Variation of pile capacity with respect to pile length is shown in the above figure. The figure shows that pile capacity increases with the increase of diameter and length of pile.

The values of average pile capacity of Khulna sub-soil are shown in the following table:

Table-5: Average Ultimate Pile Capacity of Pre-cast Pile in Khulna City by Conventional Method

Length of the Pile(ft)	Average Ultimate Pile Capacity by Conventional Method(KN)		
	Dia(12")	Dia(18")	Dia(24")
40	571.07	1185.51	1870.28
45	589.52	1203.36	1926.44
50	660.49	1319.39	2146.07
60	668.52	1352.95	2241.15
70	738.67	1514.40	2456.80

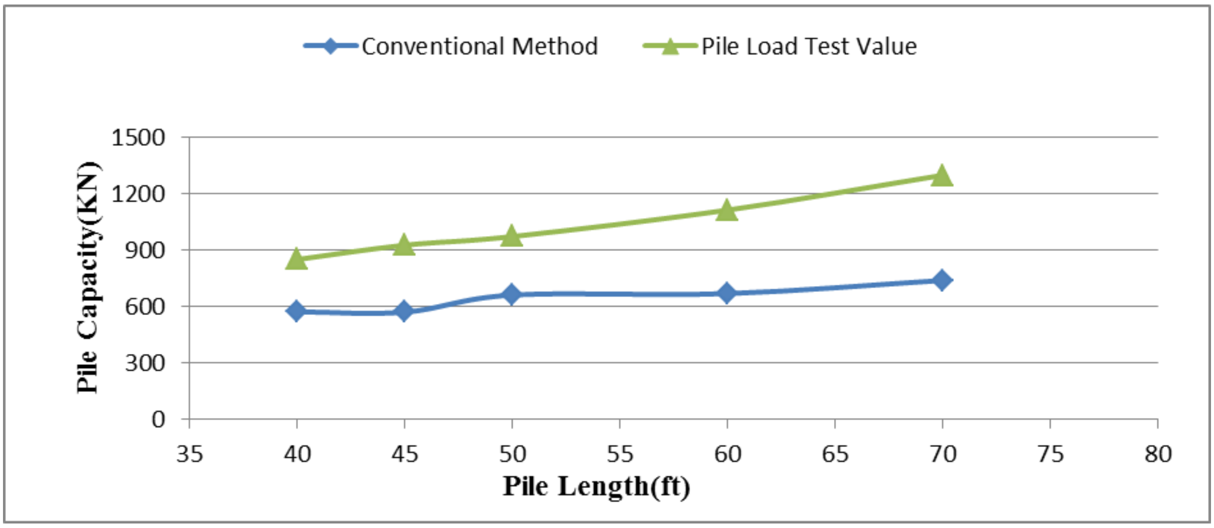


Figure-6: Comparison of Pile Capacity Obtained by Conventional Method and Pile Load Test for 12" diameter pile

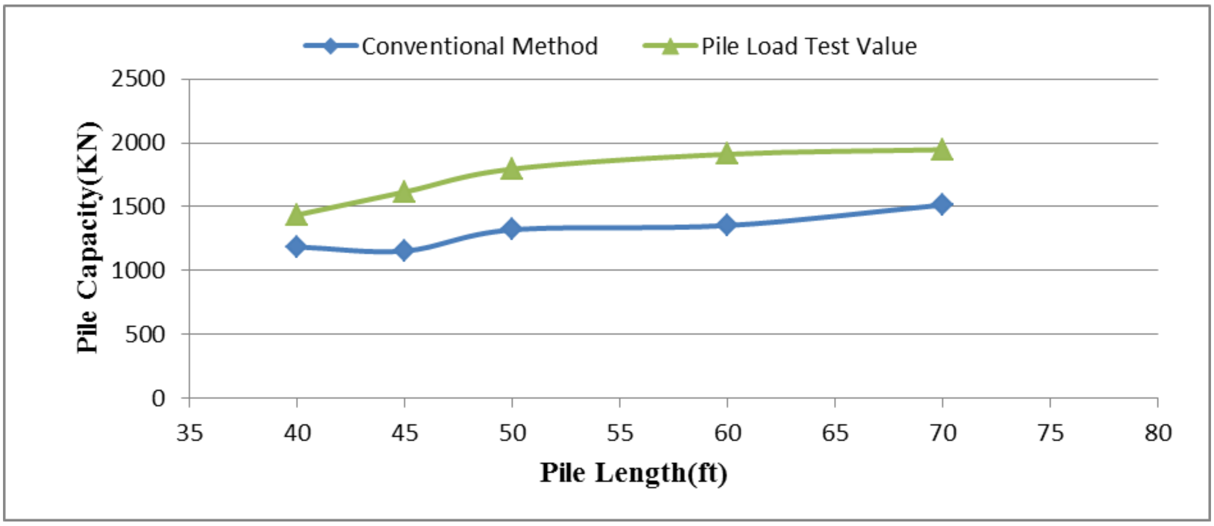


Figure-7: Comparison of Pile Capacity Obtained by Conventional Method and Pile Load Test for 18” diameter pile

It is observed that variation of pile capacity with respect to length of the pile obtained by Conventional method differs from those obtained by pile load test(Rahman,2001). This difference can be considered reasonable from practical point of view. From figure-6 and figure-7, it is observed that Conventional method gives lower value than pile load test. The results obtained from this study can be used for the design of pile in those cases where the owners are going to use pile foundation for small project without any sub-soil investigation. Moreover, for large projects pile capacity obtained by sub-soil investigation reports can be utilized for design with confidence by comparing with these results where pile load test is not performed.

8. CONCLUSIONS

The critical feature of the Khulna soil is the existence of thick compressible organic layer (the thickness of this layer varies from 10 to 40 ft) located at a depth 5 to 20 ft from ground surface in most of the places which affects the pile capacity to a great extent. Generally the pile capacity increases rapidly at about 40ft depth from the ground level. Below this depth the pile capacity generally increases with increase of depth and sometimes pile capacity decreases with the increase of depth because of the fact that the end bearing of pile sometimes decreases at higher depth. The results obtained from this study can be used for the design of pile in those cases where the owners are going to use pile foundation for small project without any sub-soil investigation. Moreover, for large projects pile capacity obtained by sub-soil investigation reports can be utilized for design with confidence by comparing with these results where pile load test is not performed.

REFERENCES

Bowels, J.E. (1997).Foundation Analysis & Design. Fifth Edition, McGraw Hill International Editions.

Molla, S.I & Malik, A.R. (1997). "Statistical Evaluation of Bearing Capacity of Shallow Foundation and Pile Capacity in Khulna Sub-soil." Undergraduate Project Report,KUET,Khulna.

Rahman, G.M. (2001)."Study on Pile Capacity in Khulna Sub-soil." M.Sc. Engineering Thesis, Khulna University of Engineering & Technology, Khulna, Bangladesh.

Singh, A. & Chowdhury, G.R. (1994)."Soil Engineering (Theory & Practice). Vol-1, Fundamental and General Principles, Third Edition, CBS Publisher and Distributors.

Teng, W.C. (2012).Foundation Design.Eastern Economy Edition, Prentice Hall of India Private Limited, New Delhi-1100001.

FEM SIMULATION OF THE VISCOUS EFFECTS OF LOADING RATE ON ALBANY SAND

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ABSTRACT

In this research, effect of strain rate on the Albany sand has been studied by elastic visco-plastic constitutive model within the three component framework. Strain rate effect can be modeled by using any one of the (i) Isotach, (ii) TESRA (Temporary Effects of Strain Rate and Acceleration) or Viscous Evanescent and (iii) P & N (Positive and Negative viscosity), models of the three component framework. Usually "Isotach" is appropriate for clay and soft rock, "TESRA" is appropriate for sand and "P & N" is appropriate for sand with less angularity like Albany sand. Here in this research Triaxial Compression (TC) tests results of Albany sand at different strain rate has been modeled successfully into a commercially available package called "ABAQUS". The P&N model was implemented into a generalized elasto-plastic isotropic strain-hardening non-linear model in C++. The model is then embedded in the finite element computer program ABAQUS. ABAQUS was used for the actual analysis. In order to define P & N model, user subroutine of Abaqus "UMAT" was written in C++ and used. ABAQUS is such a robust FEM software that allows writing subroutines for describing material behavior. Generally UMAT is written in FORTRAN but in this study, the main model is written in C++ and then it is called by FORTRAN with appropriate change in ABAQUS environment file. Without spending any significant extra computational time or storage this P & N model embedded in FE code can simulate the time-dependent stress-strain behavior accurately.

Keywords: *Constitutive law, Numerical Simulation ,Three component framework, P&N model, Elasto-plastic model, UMAT*

1. INTRODUCTION

In recent decades, problems related with long-term creep deformation of sand deposit loaded with a heavy superstructure or secondary consolidation of saturated soft clay including a number of full-scale field cases have attracted the attention of Geotechnical engineers for correctly understanding and accurately evaluating the viscous properties of geomaterials. Highly non-linear relationships of soil ware the main obstacles in soil mechanics. With the development of different experimental and analytical methods, various constitutive models for defining soil behavior have been published.

For simulating the effects of material viscosity on the stress-strain behavior of geomaterial (i.e., clay, sand, gravel, and sedimentary softrock), a set of stress-strain models within the framework of the general non-linear three-component model (Di Benedetto et al., 2002 and Tatsuoka et al. 2002) have been proposed by researchers. Three basic viscosity types have been published which are (i) Isotach, (ii) TESRA (Temporary Effects of Strain Rate and Acceleration) or Viscous Evanescent, (iii) P & N (Positive and Negative viscosity).

P & N viscosity type has been used to simulate the stress-strain behaviour of Albany sand, fine silica sand from Australia. This type of viscosity is very peculiar and was found most recently.

2. EXPERIMENTAL RESULTS

From the laboratory experiments (Tatsuoka et al.2008), it was found that four poorly graded granular materials named a) corundum A (Aluminium Oxide, Al_2O_3), an artificial material (e_{max} = 1.066 & e_{min} = 0.865); b) Albany sand, a fine silica sand from Australia (e_{max} = 0.804 & e_{min} = 0.505); c) Hime gravel, a natural fine gravel from a river bed in the Yamanashi Prefecture, Japan (e_{max} = 0.759 & e_{min} = 0.515); and d) Monterey No. 0 sand, a natural fine beach sand from the USA (e_{max} = 0.860 & e_{min} = 0.550), exhibited the P & N viscosity in the drained TC tests. In this paper, the experimental results of Albany sand are the main focus.

Loose and dense cylindrical specimens of diameter of 70 mm and height of 150-155 mm ware prepared from Albany sand. The experiments were performed on air-dried specimens to keep the loading rate effects out of the effects of delayed dissipation of excess pore water pressure.

A 0.3 mm thick latex rubber disc smeared with a 0.05 mm thick silicon grease layer (Tatsuoka et al., 1984) was used at the top and bottom ends of the each specimen. An external deformation transducer and a pair of local deformation transducers (L D T s; Goto et al., 1991) which had a gauge length of about 12cm, was used to measure axial deformation. The homogeneity in the zone of before and after peak, was not possible to evaluate. The reason of this phenomenon was discussed by Tatsuoka et al. (1990) and it was showed that local share bands start developing before the pick stress state in drained plane strain compression (PSC) tests on dense sand. Locally measured axial strains were used to calculate the elastic deformation properties. Based on the modified Rowe's stress-dilatancy relation, the volume change of air dried specimen was estimated. These experiments were done using an automated triaxial apparatus (e.g., Santucci de Magistris et al., 1999).
The specimens were loaded automatically. To control the cell pressure, a high precision gear-type axial loading system driven by a servo-motor together with an electric pneumatic pressure transducer was used. By increasing the effective stress from 20 kPa toward 400 kPa, at an axial strain rate of 0.0625%/min, the isotropic compression was performed. During the isotropic compression process, to evaluate the vertical quasi-elastic Young's modulus, eight cycles of an axial strain (double amplitude) of 0.001-0.003% were applied at p'=50,100,200 and 300 kPa.

Figure 1 illustrates results from three sets of drained TC tests performed at largely different constant axial strain rates on dense air dried specimens of Albany sand.

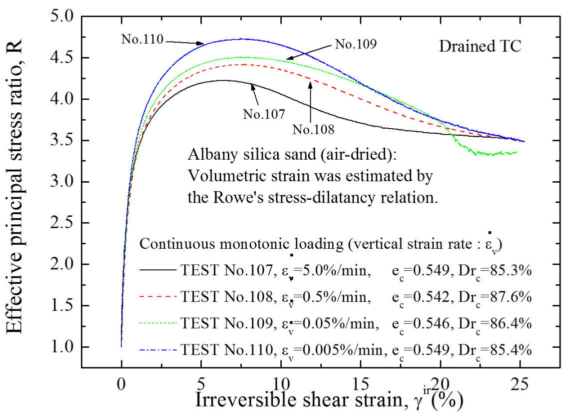

Figure 1: Results from CD TC tests at different vertical strain rates on air dried dense Albany silica sand

3. METHODOLOGY

The modelling of stress-strain behavior of geomaterial is very challenging as stress-strain behavior is highly non-linear. Development of FEA finds a way to solve the boundary value problem with highly non-linear material property. There are many commercially available FEM software now-a-days. Among them Abaqus is a robust software that allows user to model their own material model using user subroutine. But the challenge arises when user wants to write their material model in other language rather than FORTRAN. In this study, this challenge has been successfully handled as the user subroutine for material model has been written in C++ and used.

4. PSEUDO-ALGORITHM

Siddiquee et al. (2006) had developed the pseudo-algorithm which was the revised form of original solution technique of the DR method. Viscous effects were not included.

In "return mapping algorithm" (Ortiz and Simo, 1986), incremental elasto-plastic equations are solved at the first level of integration. Satisfying the consistency condition (abiding by the flow rule), the stress is returned to the growing yield surface. When calculating the viscous stress based on P&N model, the stress is returned to the inviscid yield surface with an incremental integration during the second level of integration when it is necessary at each step of return mapping iteration. This scheme is presented in Fig 2.

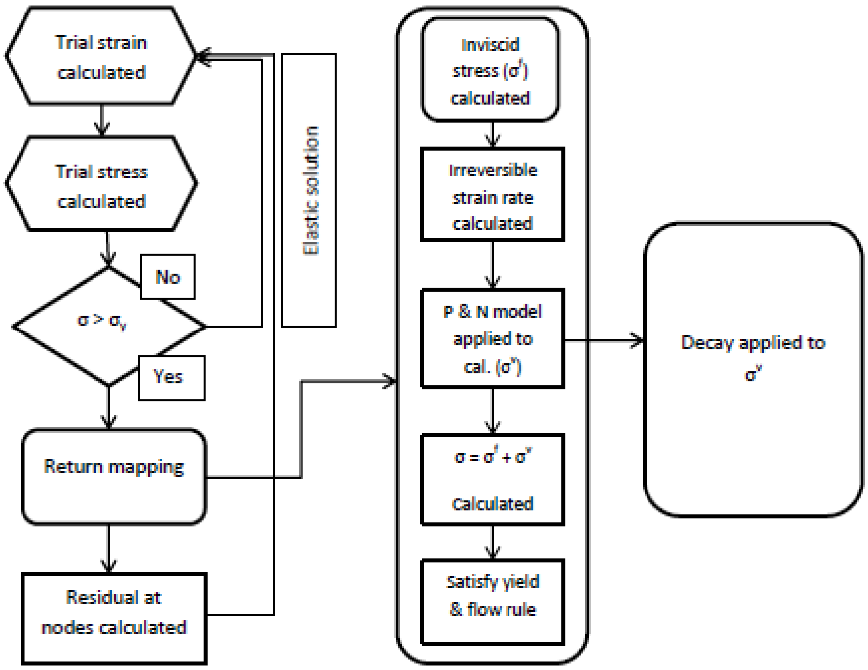

Figure 2: Implementation of the "P & N" model into a FEM code

5. OVER-ALL MODEL CALCULATION FUNCTIONS

In the user subroutine "UMAT", the stress and hardening softening parameters are calculated from strain and elastic modulus provided by Abaqus. With the updated stress and hardening softening parameters Abaqus carry out the non-linear boundary solution and provide strain and elastic modulus to "UMAT". In this process, the whole analysis is completed. The main function of user subroutine "UMAT" is UMAT_CPP. In UMAT_CPP the strain calculated by Abaqus solver is taken as input and it calculates the stress in that given moment. At the end of this function, the stress is updated to Abaqus. In this function the elastic modulus is calculated from Young's modulus and Poisson ratio. Failure surface is calculated in the function ReternMapping_. PlsticModel_ function calculates the reference curve. The function yldchk_ calculates the yield function. The calculation of Invariants is done in the function invar_. The potential function and yield function is calculated in the function yieldf.

5.1 Material Model Description

Di Benedetto et al., 2002; Tatsuoka et al., 2002. successfully simulated the rate dependent stress-strain behaviour of geomaterial observed in a number of laboratory stress-strain tests by the non-linear three-component model (Fig. 3).

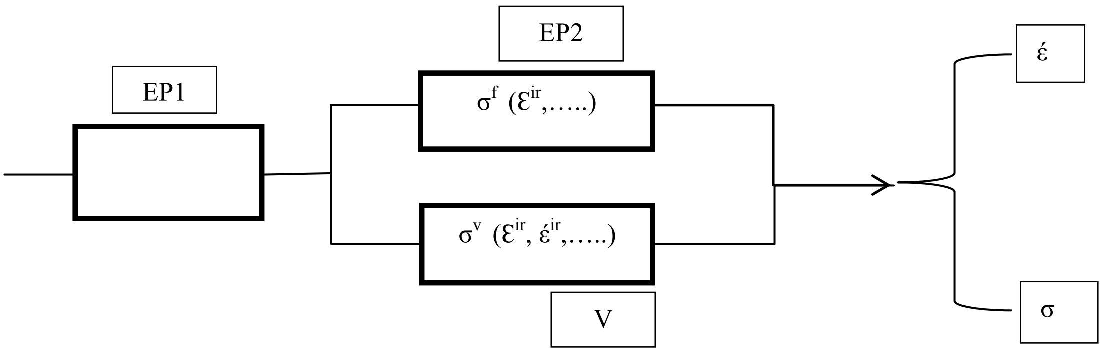

Figure 3: Non-linear three-component model (Di Benedetto and Tatsuoka, 1997; Di Benedetto et al., 2002; Tatsuoka et al., 2002)

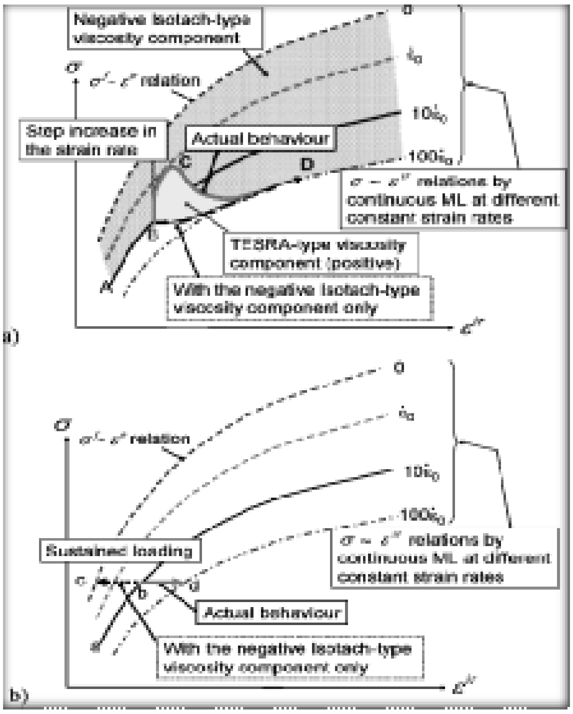

Figure 4: Illustration of P & N viscosity

Figure 4 illustrate the P&N viscosity type (Tatsuoka et al. 2008), which is defined below:
In this type of viscosity, the viscous stress increment that developed at a given moment during subsequent loading decays with an increase in instantaneous irreversible strain towards different residual values. The strength during ML at a constant e decreases with an increase in e.
In the framework of the three-component model the measured stress, σ, consists with two parts which are the inviscid stress component, σ^f, and the viscous stress component, σ^v at the same ε^{ir}. Negative isotech type is a feature of σ^v. Both positive TESRA type component and negative Isotech type component at other strain rate are the components of σ^v. This can be observed when a step increase in έ at point B during otherwise ML at a constant.The stress strain behavior should be like A→B→D if there are only negative Isotech type component. But the behavior like A→B→ C→D, instead of A→B→D is observed in poorly graded relatively round and stiff-particle granular material. A step increase in έ (B→C) results same amount of immediate positive stress increase when the viscosity type is Isotech or Combined or TESRA (Temporary Effects of strain Rate and Acceleration). After that subsequent ML at a constant έ results decrease of σ^v from a temporarily increased value (C→D) like the TESRA type. This feature was also found in the stress-strain behavior of Albany sand. For this reason P & N model is the appropriate viscosity type for simulating viscosity of Albany sand.

5.2 Computational Setup

Abaqus/CAE, or "Complete Abaqus Environment" (a recursive acronym and backronym with an obvious root in Computer-Aided Engineering) is used for both for the modelling and analysis of mechanical components and assemblies (pre-processing) and visualizing the finite element analysis result.
Full computational setup scheme described below.
G1: Installation of the finite element software ABAQUS, FORTRAN compiler and C++ compiler.
G2: Change in windows environment variable to make FORTRAN and C++ compiler available to CMD.
G3: Change in ABAQUS environment file to make .lib and .dll file available for ABAQUS.
G4: Run the verification exe to check all components are compatible with each other.
The material model code was written in C++ then it was compiled to .dll using C++ compiler. From .dll, using CMD and C++ compiler the .lib file was created. The finite element model was created using Abaqus/CAE and using FORTRAN the material model was called and performed the analysis.

6. DETAILS OF THE MODEL

6.1 Parameters used

Kongkitkul et al. (2008) described various aspects of the simulation. Tatsuoka et al. (2008) represented simulation parameters for the stress-strain behaviour exhibiting the P & N viscosity.

Table 1: Viscosity parameters used to analyse the CD triaxial tests

Material	**Strain parameter**	**β: test results**	**Parameters in the viscosity function**			**Back-calculated by fitting**		**Decay parameter**	**Viscosity type parameter, Θ**			
			α	**m**	$\dot{\varepsilon}_r^{ir}$	**b**	**β: from b**		Θ_{ini}	Θ_{end}	**c**	ε_Θ^{ir}: **%**
Albany sand	Irreversible Shear strain	0.0195	0.24	0.04	1.0E-5(%/s)	0.00827	0.0190	1.0E-03	-0.3	-1.0	1.0	12

6.2 Elasto-plastic framework

The present study is done using the generalized elasto-plastic isotropic strain-hardening and softening model which takes into account strain localization associated with shear banding by introducing a characteristic width of shear band in the additive elasto-plastic decomposition of strain (Tatsuoka et al., 1993). The yield function is used as follows:

$$\Phi = -\eta I_1 + \frac{1}{g(\theta)}\sqrt{J_2} - K \tag{1}$$

The above equation is used as the growth function of the yield surface of the generalized Mohr-Coulmb type. Where I_1 is the first stress invariant (i.e., hydrostatic stress component, positive in compression); and J_2 is the second deviatoric stress invariant (i.e., the deviatoric stress). Siddiquee et al. (1999, 2001a and b) had explained in detail about the growth function.

The plastic potential function, ψ, is defined as;

$$\psi = -\alpha' I_1 + \sqrt{J_2} - K \tag{2}$$

This plastic potential function, of the Drucker-Prager type, is similar to the yield function except that g(Θ) in equation 1. Here in the analysis, stress dependent elastic parameters are used

7. RESULTS AND DISCUSSIONS

Four different strain-rate experimental results are simulated successfully in this study.

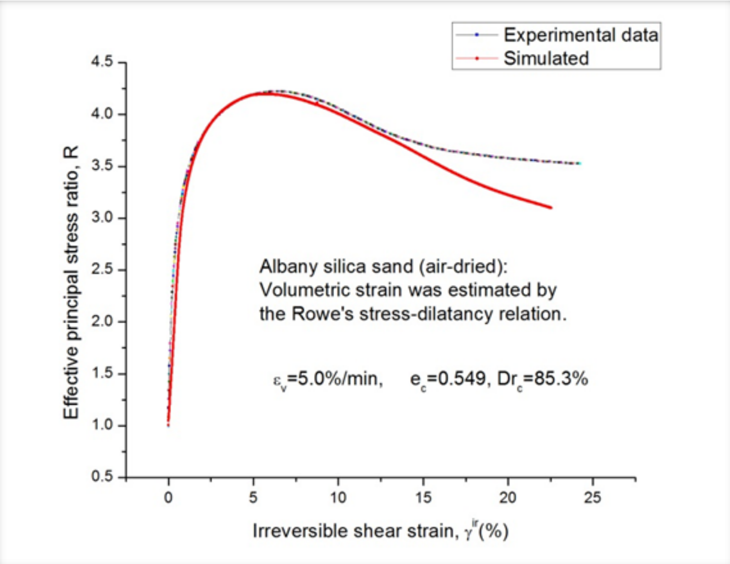

Figure 5: Experimental and simulated curve of Effective principal stress, R vs Irreversible shear strain at a vertical strain rate 5.0%/min

In Fig. 5, the simulated curve has been compared with the experimental data of TC test at a vertical strain rate 5.0% / min. In this simulation, the peak effective principal stress was 4.22 at irreversible shear strain 6.6%. The simulated curve is largely deviated from experimental curve after irreversible shear strain 13.4%.

In Fig. 6, the simulated curve has been compared with the experimental data of TC test at a vertical strain rate 0.5%/min. In this simulation, the peak effective principal stress was 4.4 at irreversible shear strain 8.2%. The simulated curve is largely deviated from experimental curve after irreversible shear strain 5.51%.

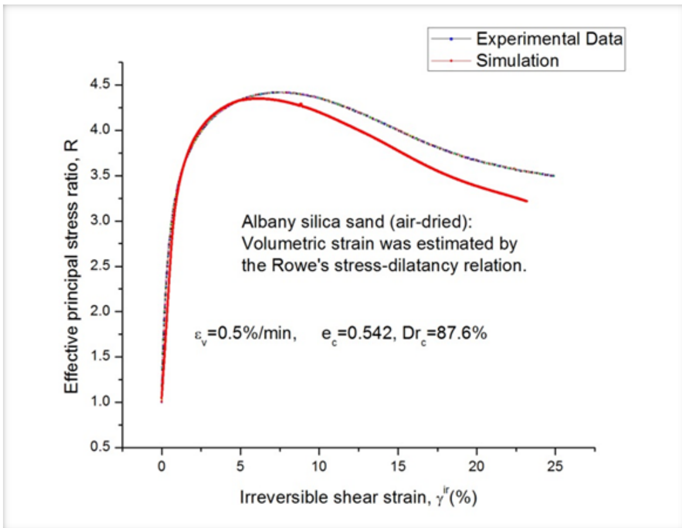

Figure 6: Experimental and simulated curve of Effective principal stress, R vs Irreversible shear strain at a vertical strain rate 0.5%/min

In Fig. 7, the simulated curve has been compared with the experimental data of TC test at a vertical strain rate 0.05%/min. In this simulation the peak effective principal stress was 4.5 at irreversible shear strain 7.5%. The simulated curve is largely deviated from experimental curve after irreversible shear strain 9.4%.

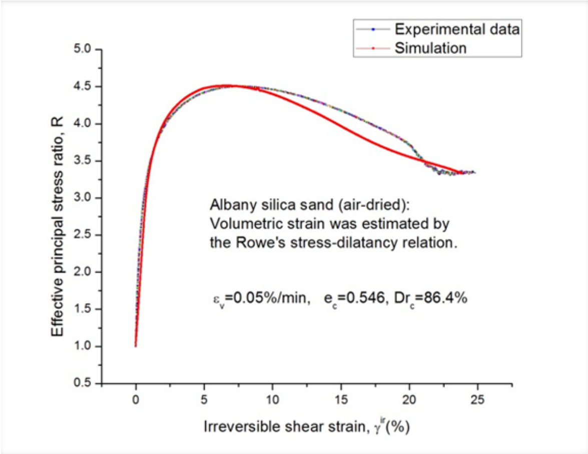

Figure 7: Experimental and simulated curve of Effective principal stress, R vs Irreversible shear strain at a vertical strain rate 0.05%/min

In Fig. 8, the simulated curve has been compared with the experimental data of TC test at a vertical strain rate 0.005%/min. In this simulation, the peak effective principal stress was 4.7 at irreversible shear strain 7.15%. As this curve is accounted as base, the simulated and experimental curve is nearly same.

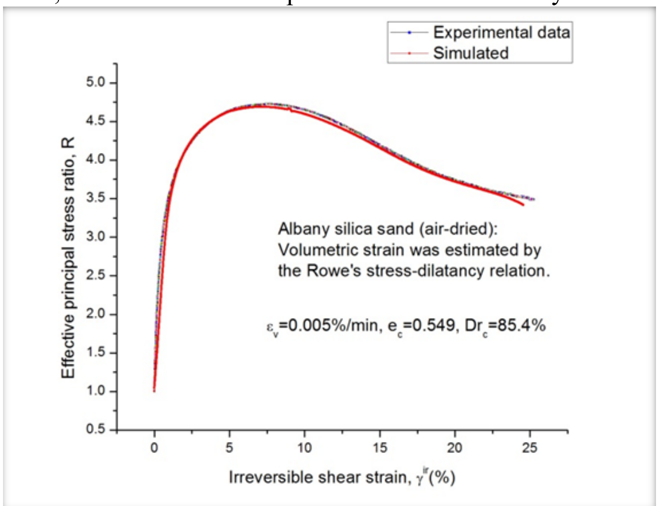

Figure 8: Experimental and simulated curve of Effective principal stress, R vs Irreversible shear strain at a vertical strain rate 0.005%/min

8. CONCLUSIONS

The elastic visco-plastic analysis of the TC tests is not a new topic. However, this study was devoted to model the experimentally observed findings in a very realistic and simplified way. The yield function was Mohr-Coulomb type and plastic potential was Drucker-Prager type. The following conclusions can be drawn from this study.

1. Effect of strain rate on the Albany sand has been studied by combination of elastic visco-plastic constitutive law and three component framework.

2. TC tests results of Albany sand at different strain rate has been modelled successfully into a commercially available package called “ABAQUS”.

3. The P&N model was implemented into a generalized elasto-plastic isotropic strain-hardening non-linear model in C++. The model is then embedded in the finite element computer program ABAQUS.

4. For small strain the experimental data was successfully simulated but problem was associated with large deformation. The simulated curve deviated more or less from 7.5 % irreversible shear strain. The deviation was higher for vertical strain rate at 0.3%/min.

5. A FORTRAN compiler is required to compile and link user subroutines for Abaqus. But it also allows writing user subroutines in languages other than FORTRAN with the FORTRAN compiler specified and a compiler for that language. It is needed to call routines in that language from FORTRAN.

6. In this study, user subroutine was written in C++ rather than FORTRAN. With the help of C++ compiler .dll and .lib file was created and they were placed in appropriated destination and the environmental file was updated. This made possible to call the user subroutine from FORTRAN and simulation of stress-strain behaviour of Albany sand.

7. Without spending any significant extra computational time or storage this P & N model embedded in FE code can simulate the time-dependent stress-strain behaviour accurately.

9. ACKNOWLEDGEMENTS

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REFERENCES

Siddiquee, M.S.A., Tatsuoka, T., and Tanaka, T. (2006). “FEM simulation of the viscous effects on the stress-strain behaviour of sand in plane strain compression” Soils and Foundations, Vol. 46(1), 99-108.

Tatsuoka, T. Di Benedetto, H., Enomoto, T., Kawabe, S. and Kongkitkul, W. (2008). “Various viscosity types of geomaterials in shear and their mathematical expression” Soils and Foundations, Vol. 48(1), 41-60.

Di Benedetto, H., Tatsuoka, F. and Ishihara, M. (2002). “Time-dependant shear deformation characterestics of sand and their constitutive modelling” Soils and Foundations, 42(2), 1-22.

Di Benedetto, H., Tatsuoka, F. (1997). “Small strain behaviour of geomaterials: modelling of strain effects” Soils and Foundations, Vol. 37(2), 127-138.

Santucci de Magistris, F.,Koseki, J., Amaya, M., Hamaya, S., Sato, T. and Tatsuoka, F. (1999). “A triaxial testing system to evaluate stress-strain behaviour of soils for wide range of strain and strain rate” Geotechnical Testing Journal, ASTM, Vol 22(1), 44-60.

Tatsuoka, F., Molenkamp, F., Torii, T. and Hino, t. (1984). “Behaviour of lubrication layers of platens in element tests” Soils and Foundations, Vol 24(1), 113-128

Goto, S., Tatsuoka, F., Shibuya, S., Kim, Y.-S. and Sato, T.(1991). “A simple gauge for local small strain measurements in the laboratory” Soils and Foundations, Vol 31(1), 169–180.

Tatsuoka, F., Nakamura, S., Huang, C.-C. and Tani, K. (1990). “Strength anisotropy and shear band direction in plane strain tests on sand”. Soils and Foundations, Vol 30(1), 35–54.

Ortiz, M. and Simo, J. C. (1986). “An analysis of a new class of integration algorithms for elasto-plastic constitutive relations” *Int. J. Num. Meth. Engg.,* vol 23, 353-366.

Tatsuoka, F., Di Benedetto, H., Kongkitkul, W. Kongsukprasert, L. and Nishi, T. (2008). “Modelling of ageing effects on the elastoviscoplastic behaviour of geomaterial” Soils and Foundations, 48(2) 155-174.

Tatsuoka, F., Siddiquee, M.S.A., park, C.-S., Sakamooto, M. and Abe, F. (1993). “Modelling stress-strain relations of sand” Soils and Foundations, Vol 33(2), 60-81.

Siddiquee, M. S. A., Tatsuoka, F., Tanaka, T., Tani, K., yoshida, K. and Morimoto, T. (2001) “ Model tests and FEM simulation of some factors affecting the bearing capacity of footing on sand” Soils and Foundations, Vol 41(2), 53-76.

Siddiquee, M. S. A., Tatsuoka, F. (2001) “Modelling time dependant stress-strain behaviour of stiff geomaterials and its applications” *Proc. 10th Int. Conf. Comput. Math. Adv. Geomech.(IACMAG),* January 7-12, Tucson, Arizona.

Siddiquee, M. S. A., Tatsuoka, F., Tanaka, T., Tani, K. and Morimoto, T. (1999) “ FEM simulation of scale effect in bearing capacity of strip footing on sand” Soils and Foundations, Vol 34(4), 91-109.

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ARTIFICIAL NEURAL NETWORK APPROACH FOR THE PREDICTION OF UPLIFT CAPACITY OF ANCHOR FOUNDATION

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ABSTRACT

Anchors are often used for the foundations to resist uplift forces. The uplift capacity of anchor foundation depends on its embedment, shape, size, inclination and the relative density of soils. The uplift capacity of such foundation is an important issue in its application, and reliable methods of predicting the capacity are required to produce effective design. In this paper a back-propagation neural network model is developed to predict the uplift capacity of anchor foundation. A database containing the results from number of model test and centrifuge tests is used. The result of this study indicates that the ANNs serve as a reliable and simple predictive tool for the uplift capacity of anchor foundation. The neural network predictions are compared with those calculated using traditional methods.

Keywords: *Anchor, Uplift, Embedment, Shape, Size, Inclination, ANN.*

1. INTRODUCTION

There are many examples in civil engineering design where the requirement to resist pull-out forces acting on foundations embedded in the ground has to be met. In such cases an attractive and economic design solution may be achieved by the use of tension members that are fixed to the foundation and embedded in the ground to sufficient depth that they can be resist the pull out forces with adequate safety. These members are referred to as anchors. Since anchor foundations having different embedment ratios, sizes, and shapes are widely used for towers, bridges, and various other structures in order to resist tensile (upward) forces, many studies have focused on pulling out these types of foundations. To find the design methodology for such foundations, most of the researchers (Majer 1968; Mors 1959; Balla 1961; Baker and Kondner 1966; Meyerhof and Adams 1968; Vesic 1971; Vermeer and Sutjiadi 1985; Murray and Geddes 1987; Merifield and Sloan 2006, Rowe and Davies 1982, Sakai and Tanaka 2007, etc.) mainly concentrated on design approach, based on limit equilibrium, cavity expansion, limit state theory, or displacement based elasto-plastic finite element method (FEM). For the safe and economical design of the anchor foundation, it is very necessary to provide an accurate numerical model with the geotechnical engineers. In this regard, Md. Rokonuzzaman and Toshinori Sakai (2013) employed 3D FE model to analyze the rectangular anchor problems. But these are very high level research task which are very complicated for understanding of the professional foundation engineers. By considering all of these factors the prediction of uplift capacity of anchor rectangular foundation can be easily done by artificial neural analysis (ANN). During the last two decades several researchers have developed effective modelling tools such as ANNs and fuzzy rule based approaches in engineering. In the recent past, ANNs have been applied to many geotechnical engineering problems, including the prediction of the bearing capacity of piles (Goh, 1995a; 1996b), settlement predictions (Shahin et al., 2000), liquefaction potential (Goh ,1996a) ,developing engineering correlations between various soil parameters(Goh ,1995a; 1995c)and so on. This indicates that ANNs can be used for both prediction and forecasting of events. The major advantage of ANNs is that they can be updated easily as and when new data become available that eliminates the need for a specialist to reanalyze the old and new data, update the old design aids or equations and/or propose new equations.

This study has been done to investigate the feasibility of the ANN technique for predicting the uplift capacity of anchor foundations in cohesion less soils and to provide an executable program of the developed ANN model for routine use in practice. The effect of ANN geometry and some internal parameters on the performance of ANN models will be focused.

2. OVERVIEW OF ARTIFICIAL NEURAL NETWORKS

Artificial Neural Network is a system that mimics the human brain and therefore a great deal of the terminology is borrowed from neuroscience. The most basic element of the human brain is a specific type of cell, which provides us with the abilities to remember, think and apply previous experience to our every action. These cells are known as neurons as shown in Figure-1, each of these neurons can connect with up to 200,000 other neurons. The power of brain comes from the numbers of these basic components and the multiple connections between them.

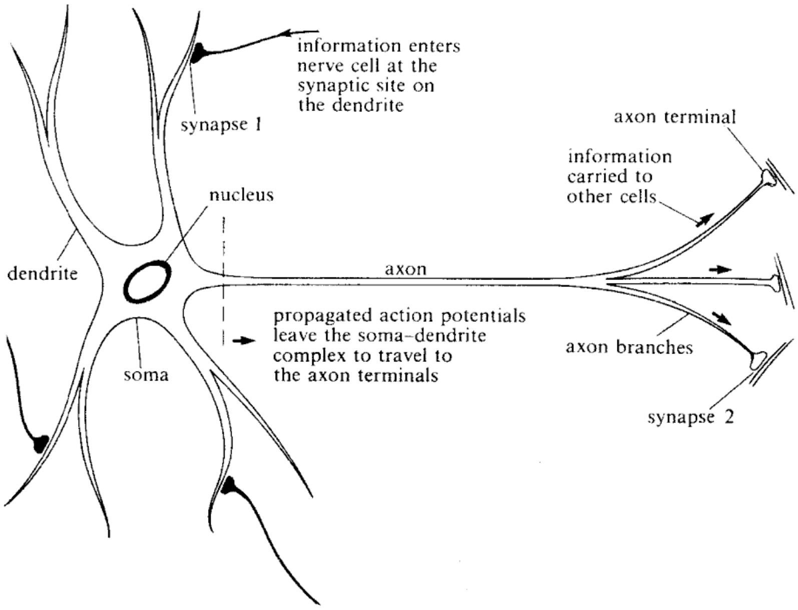

Figure -1: A biological neuron and its components

All natural neurons have four basic components, which are dendrites, soma, axon, and synapses. Basically a biological neuron receives inputs from other sources, combines them in some way, performs a generally nonlinear operation on the result, and then output the final result. A comprehensive description of ANNs is beyond the scope of this paper. Many authors have described the structure and operation of ANNs (e.g. Hecht-Nielsen 1990; Maren et al. 1990; Zurada 1992; Fausett 1994; Ripley 1996). A typical structure of ANNs consists of a number of processing elements (PEs), or nodes, that are usually arranged in layers: an input layer, an output layer and one or more hidden layers (Figure-2).

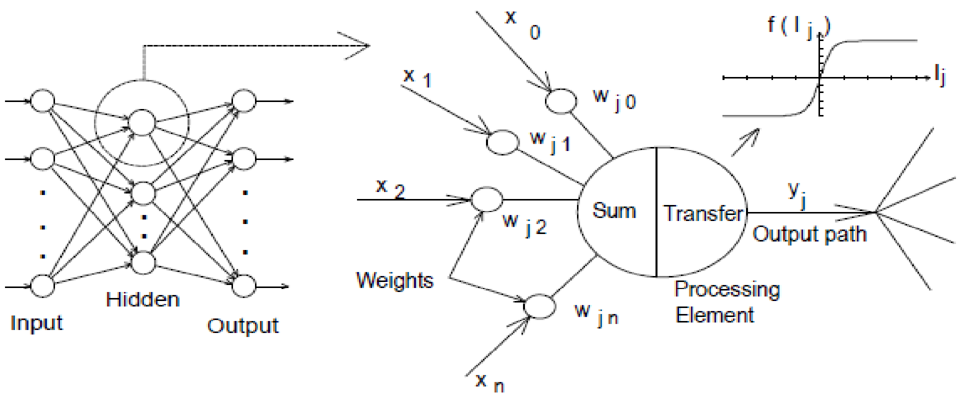

Figure -2: A processing unit in a Neural Network

The input from each PE in the previous layer (x_i) is multiplied by an adjustable connection weight (w_{ji}). At each PE, the weighted input signals are summed and a threshold value (θ_j) is added. This combined input (I_j) is then passed through a non-linear transfer function (f (.)) to produce the output of the PE (y_j). The output of one PE provides the input to the PEs in the next layer. This process is summarized in Equations 1 and 2 and illustrated in Figure 1. Once PE propagates to the output layer, the network error is computed as shown in Equation 3.

$$I_j = \sum W_{ji} X_i + \theta_j \quad \text{Summation} \tag{1}$$

$$y_j = f(I_j) \quad \text{Transfer} \tag{2}$$

$$E = \frac{1}{2}\sum_j (y_j - d_j)^2 \quad \text{Error} \tag{3}$$

The propagation of information in ANNs starts at the input layer where the input data are presented. The network adjusts its weights on the presentation of a training data set and uses a learning rule to find a set of weights that will produce the input/output mapping that has the smallest possible error. This process is called "learning" or "training". The typical and widely used first-order gradient descent technique Levenberg Marquardt back-propagation algorithm has the following steps:

1. It computes how fast the error changes as the activity of an output unit are changed. This error derivative (EA) is the difference between the actual and the desired activity (equation 4).

$$EA_j = \frac{\partial E}{\partial y_j} = y_j - d_j \tag{4}$$

2. It then computes how fast the error change as the total input received by an output unit is changed. This quantity (EI) is the answer from step 1 multiplied by the rate at which the output of a unit changes as its total input is changed (equation 5).

$$EI_j = \frac{\partial E}{\partial x_j} = \frac{\partial E}{\partial y_j} \times \frac{d y_j}{d x_j} = EA_j y_j (1 - y_j) \tag{5}$$

3. It then computes how fast the error changes, as a weight on the connection into an output unit is changed. This quantity (EW) is the answer from step 2 multiplied by the activity level of the unit from which the connection emanates (equation 6).

$$EW_{ij} = \frac{\partial E}{\partial w_{ij}} = \frac{\partial E}{\partial x_j} \times \frac{d x_j}{d w_{ij}} = EI_j y_j \tag{6}$$

4. It then computes how fast the error change as the activity of a unit in the previous layer is changed. This crucial step allows back propagation to be applied to multilayer networks. When the activity of a unit in the previous layer changes, it affects the activities of all the output units to which it is connected. So to compute the overall effect on the error, we add together all these separate effects on output units. But each effect is simple to calculate. It is the answer in step 2 multiplied by the weight on the connection to that output unit (equation 7).

$$EA_j = \frac{\partial E}{\partial y_j} = \sum_j \frac{\partial E}{\partial x_j} \times \frac{d x_j}{d y_j} = \sum_j EI_j W_{ij} \tag{7}$$

By using steps 2 and 4, we can convert the EAs of one layer of units into EAs for the previous layer. This procedure can be repeated to get the EAs for as many previous layers as desired. Once the EA of a unit is known, we can use steps 2 and 3 to compute the EWs on its incoming connections.

5. It then advances to compute the H matrix (equation 8) and the gradient (equation 9). These are necessary in order to approach second-order training speed without having to compute the Hessian matrix. The performance function has the form of a sum of squares (MSE) and as such the Hessian matrix can be approximated as given in equation 8 and then the gradient can be computed using equation 9 as follows:

$$H = J^T J \tag{8}$$

$$g = J^T e \tag{9}$$

Where, J is the Jacobian matrix that contains first derivatives of the network errors with respect to the weights and biases, and e is a vector of network errors. The Jacobian matrix can be computed through a standard back propagation technique that is much less complex than computing the Hessian matrix.

6. Finally, the Levenberg Marquadt- lm algorithm uses this approximation to the Hessian matrix in the following Newton-like update (equation 10).

$$x_{k+1} = x_k - [J^T J + \mu I]^{-1} J^T e \qquad (10)$$

Where x_{k+1} is the updated value of the network weight or bias and x_k is the current weight or bias value. When the scalar μ is zero, this is just Newton's method, using the approximate Hessian matrix. When μ is large, this becomes gradient descent with a small step size. Newton's method is faster and more accurate near an error minimum, so the aim is to shift toward Newton's method as quickly as possible. Thus, μ is decreased after each successful step (reduction in performance function) and is increased only when a tentative step would increase the performance function. In this way, the performance function will always reduce in successive iterations of the algorithm (Buhari and Adamu, 2012).

Once the training phase of the model has been successfully accomplished, the performance of the trained model has to be validated using an independent testing set. Details of the ANN modelling process and development are beyond the scope of this paper and are given elsewhere (e.g. Moselhi et al. 1992; Flood and Kartam 1994; Maier and Dandy 2000).

As described above, ANNs learn from data examples presented to them and use these data to adjust their weights in an attempt to capture the relationship between the model input variables and the corresponding outputs. Consequently, ANNs do not need any prior knowledge about the nature of the relationship between the input/output variables, which is one of the benefits that ANNs have compared with most empirical and statistical methods .The ANN modelling philosophy is similar to a number of conventional statistical models in the sense that both are attempting to capture the relationship between a historical set of model inputs and corresponding outputs. For example, suppose a set of *x*-values and corresponding *y*-values in 2 dimensional space, where $y = f(x)$. The objective is to find the unknown function *f*, which relates the input variable *x* to the output variable *y*. In a linear regression model, the function *f* can be obtained by changing the slope tanφ and intercept β of the straight line in Figure-3, so that the error between the actual outputs and outputs of the straight line is minimized. The same principle is used in ANN models. ANNs can form the simple linear regression model by having one input, one output, no hidden layer nodes and a linear transfer function (Figure-4). The connection weight *w* in the ANN model is equivalent to the slope tanφ and the threshold θ is equivalent to the intercept β, in the linear regression model. ANNs adjust their weights by repeatedly presenting examples of the model inputs and outputs in order to minimize an error function between the historical outputs and the outputs predicted by the ANN model.

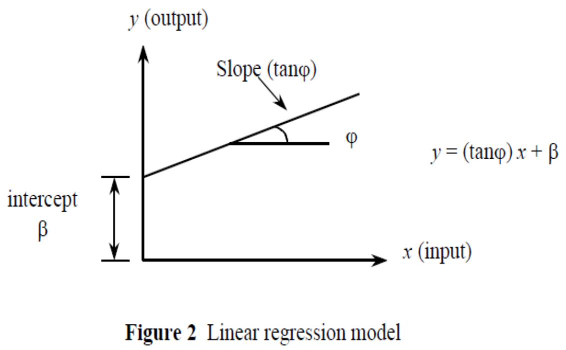

Figure-3: Simple linear regression model

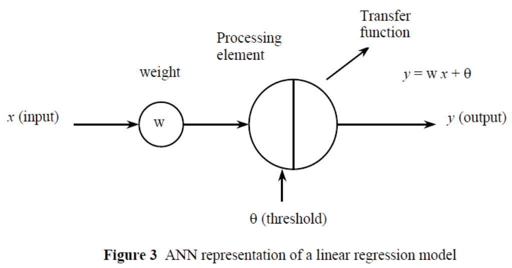

Figure-4: ANN representation of linear regression model

If the relationship between *x* and *y* is non-linear, regression analysis can only be successfully applied if prior knowledge of the nature of the non-linearity exists. On the contrary, this prior knowledge of the nature of the non-linearity is not required for ANN models. In the ANN model, the degree of non-linearity can be also changed easily by changing the transfer function and the number of hidden layer nodes. In the real world, it is likely to encounter problems that are complex and highly non-linear. In such situations, traditional regression analysis is not adequate (Gardner et al., 1998). In contrast, ANNs can be used to deal with this complexity by changing the transfer function or network structure, and the type of non-linearity can be changed by varying the number of hidden layers and the number of nodes in each layer. In addition, ANN models can be upgraded from univariate to multivariate by increasing the number of input nodes.

3. DEVELOPMENT OF NEURAL NETWORK MODEL

The steps for developing ANN models include the determination of model inputs and outputs, division and pre-processing of the available data, the determination of appropriate network architecture, optimization of the connection weights (training), stopping criteria, and model validation. The ANN toolbox of MATLAB (The MATHWORKS 2011 R2011a) is used to simulate the problem.

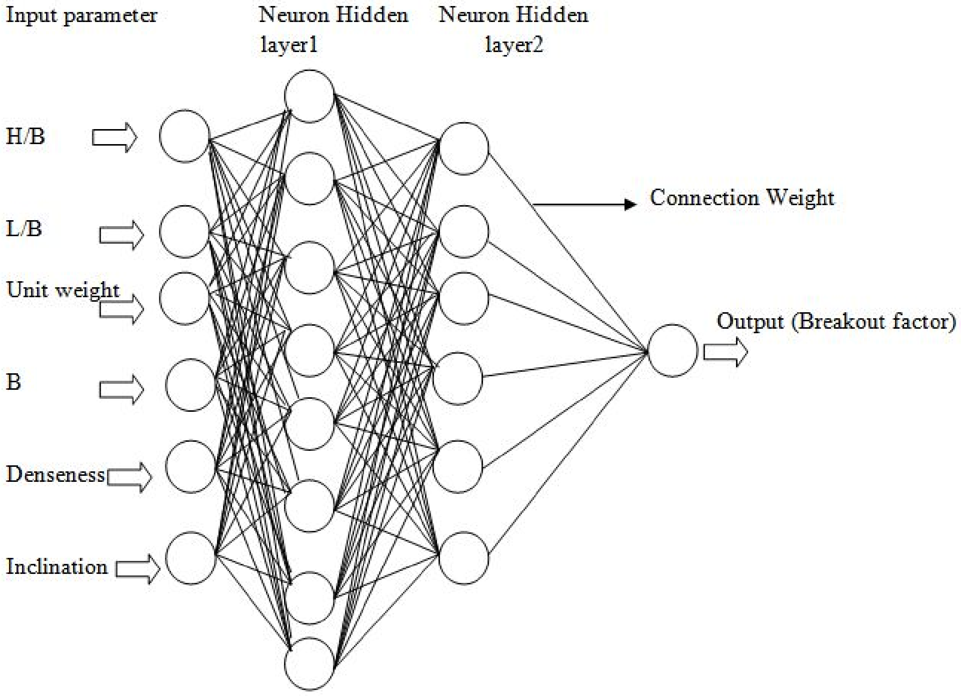

Figure-5: A typical ANN model architecture

3.1 Database

In order to calibrate and validate the ANN model, data were collected from the literature and include the laboratory model tests on the uplift capacity of anchor foundation. The data cover a wide range of variation in anchor dimensions and soil properties. Few data are available for clay soil and therefore sands are considered only in this study. The database comprises a total of 233 individual cases reported by Rokonuzzaman and Toshinori Sakai (2013), Murray & Geddes (1987), Dickin (1988), Frydman and Shahman (1989),Das and Selley (1975), Rowe &Davis (1982), Balla (1961), Overseen(1981), Smith (1962),Dickin& Leung(1983).

3.2 Model Input and Output

A thorough understanding of the factors affecting the uplift capacity of anchor is needed in order to obtain accurate breakout factor prediction. The main factors affecting anchor capacity includes: width (B), embedment (H/B), aspect ratio (L/B), inclination (α), soil denseness (Dr) and soil unit weight (γ). The aforementioned factors are presented to the ANN as model input variables. Dimensionless breakout factor (Nq) is the single output variable. Figure 5 shows a typical architecture of an ANN model.

3.3 Data Division and Pre-processing

It is common practice to divide the available data into two subsets; a training set, to construct the neural network model, and an independent validation set to estimate model performance in the deployed environment. However, dividing the data into only two subsets may lead to model over fitting. As a result, and as discussed later, cross validation is used as the stopping criterion in this study and, consequently, the database is randomly divided into three sets: training, testing, and validation. In total, 80% of the data are used for training and 20% are used for validation. The training data are further divided into 70% for the training set and 30% for the testing set. The dividing process was carried out randomly between the three sets and each dataset has been statistically examined to ensure that it covers the range of input parameters. Like all empirical models, ANNs are unable to extrapolate beyond the range of their training data. Consequently, in order to develop the best possible model, given the available data, all patterns that are contained in the data need to be included in the training set. Similarly, since the test set is used to determine when to stop training, it needs to be representative of the training set and should therefore also contain all of the patterns that are present in the available data. If all the available patterns are used to calibrate the model, then the most challenging evaluation of the generalization ability of the model is if all of the patterns are also part of the validation set. Consequently, it is essential that the data used for training, testing, and validation represent the same population (Masters, 1993). In order to achieve this in the present study, several random combinations of the training, testing, and validation sets are tried until three statistically consistent data sets are obtained. The statistical parameters considered include the mean, standard deviation, minimum, maximum, and range. The data ranges used for the ANN model variables are given in Table -1.Once the available data have been divided into their subsets, it is important to preprocess the data to a suitable form before they are applied to the ANN. Preprocessing the data by scaling them is important to ensure that all variables receive equal attention during training. The output variables have to be scaled to be commensurate with the limits of the transfer functions used in the output layer. Scaling the input variables is not necessary but is always recommended (Masters, 1993). In this work, the input and output variables are scaled between 0.0 and 1.0, as the sigmoidal transfer function is used in the output layer.

Table-1: Statistics of the database

Model variables	Minimum	Average	Maximum	Range	Standard Dev.
Embedment, H/B	1	4.3	9.3	8.3	2.2
Aspect ratio, L/B	1	6.2	13.3	12.3	3.4
Size, B (mm)	10	40.2	91.0	81.0	17.4
Unit wt.(kN/m^3)	14.2	15.4	17.1	2.8	0.8
Denseness	Loose	Medium	dense	--	--
Inclination	21	53.8	90	69	22.8
Breakout factor (N_q)	0.4	7.5	40.1	39.6	7.2

3.4 Model Architecture

Determining the network architecture is one of the most important and difficult tasks in the development of ANN models. It requires the selection of the number of hidden layers and the number of nodes in each of these. It has been shown that a network with one hidden layer can approximate any continuous function, provided that sufficient connection weights are used (Hornik et al. 1989).Consequently, one hidden layer is used in this study. The number of nodes in the input and output layers are restricted by the number of model inputs and outputs. The input layer of the ANN model developed in this work has six nodes, one for each of the model inputs i.e., width (B), embedment (H/B), aspect ratio (L/B), inclination (α), soil denseness (D_r) and soil unit weight (γ). The aforementioned factors are presented to the ANN as model input variables. The output layer has only one node representing the measured value of breakout factor (N_q).In order to obtain the optimum number of hidden layer nodes, it is important to strike a balance between having sufficient free parameters (weights) to enable representation of the function to be approximated, and not having too many so as to avoid overtraining and to ensure that the relationship determined by the ANN can be interpreted in a physical sense. Overtraining is not an issue in this study, as cross validation is used as the stopping criterion. However, as just discussed, physical interpretation of the connection weights is important, and hence the smallest network that is able to map the desired relationship should be used. In order to determine the optimum network geometry, ANNs with two, five, ten, and 15 hidden layer nodes are trained.

3.5 Weight Optimization (Training)

The process of optimizing the connection weights is known as ‘‘training’’ or ‘‘learning”. As mentioned previously, this is equivalent to the parameter estimation phase in conventional statistical models. The aim is to find a global solution to what is typically a highly nonlinear optimization problem. As the prediction of uplift capacity of anchor foundations in cohesionless soils does not involve any time-related parameter components, feed-forward, rather than recurrent, networks are used. The method most commonly used for finding the optimum weight combination for feed-forward neural networks is the back-propagation algorithm (Rumelhart et al. 1986), which is based on first-order gradient descent optimization method. Feed-forward networks trained with the back-propagation algorithm have already been applied successfully too many geotechnical engineering problems (e.g., Goh 1994; Najjar and Basheer 1996), and are thus used in this work. Details of the back-propagation algorithm are beyond the scope of this paper and can be found in many publications (e.g., Fausett

1994).In this study, the general strategy adopted for finding the optimal parameters that control the training process is as follows. For each trial number of hidden layer nodes, random initial weights and biases are generated. The neural network is then trained with different combinations of momentum terms and learning rates in an attempt to identify the ANN model that performs best on the testing data. Since the back-propagation training algorithm uses a first-order gradient descent technique (Levenberg-Marquardt) to adjust the connection weights, it may get trapped in a local minimum if the initial starting point in weight space is unfavorable. Consequently, the model that has the optimum momentum term and learning rate is retrained a number of times with different initial weights and biases until no further improvement occurs.

3.6 Stopping Criteria

Stopping criteria are those used to decide when to stop the training process. They determine whether the model has been optimally or sub optimally trained. As described earlier, the cross validation technique (Stone 1974) is used in this work as the stopping criterion, as it is considered to be the most valuable tool to ensure that over fitting does not occur (Smith 1993) and as sufficient data are available to create training, testing, and validation sets. The training set is used to adjust the connection weights. The testing set measures the ability of the model to generalize, and the performance of the model using this set is checked at many stages of the training process, and training is stopped when the error of the testing set starts to increase. The testing set is also used to determine the optimum number of hidden layer nodes and the optimum internal parameters (learning rate, momentum, and initial weights).

3.7 Model Validation

Once the training phase of the model has been successfully accomplished, the performance of the trained model is validated using the validation data, which have not been used as part of the model building process. The purpose of the model validation phase is to ensure that the model has the ability to generalize within the limits set by the training data, rather than simply having memorized the input–output relationships that are contained in the training data. The coefficient of determination (R^2) and the root-mean-square error (RMSE) are the main criteria that are used to evaluate the performance of the ANN models developed in this work. The coefficient of determination is a measure that is used to determine the relative correlation between two sets of variables. The RMSE is the most popular measure of error and has the advantage that large errors receive greater attention than smaller ones (Hecht-Nielsen 1990). Final model is chosen with hidden neuron of five due its bets performance (Table-2).

Table -2: Performance of different networks

Hidden neurons	Root Mean Square Error (RMSE) (Validation)	Coefficient of determination of R^2 (%)		
		Training	Testing	Validation
2	11.6377 at epoch 10	76.30	69.93	85.52
5	1.2525 at epoch 8	93.03	90.33	98.84
10	7.9851 at epoch 7	81.18	86.67	88.08
15	3.0006 at epoch 8	95.53	83.53	97.50

4. PARAMETRIC STUDY

ANN modeling provides a convenient way of conducting a parametric study. The effect of each parameter on the value of uplift capacity at a certain site is further investigated in this study (Figures 6-8). The basic methodology used is to change only one input variable at a time. From the Figure-7, it can be concluded that, with the increase of length to width ratio of anchor foundation break factor also increases. But from the Figure -8, it is seen that the width of anchor foundation has no effect on break out factor. Figure -6 indicates that, with the increase of embedment ratio (H/B), the break out factor also increases.

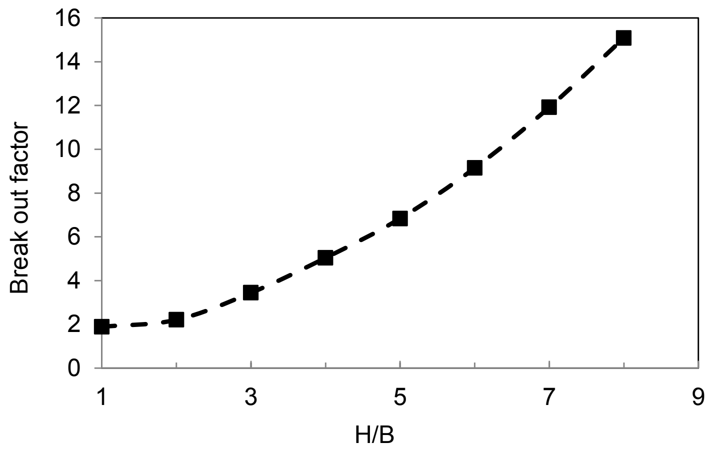

Figure-6: Pullout capacity as a function of H/B ratio (L/B=2 and B=50 mm)

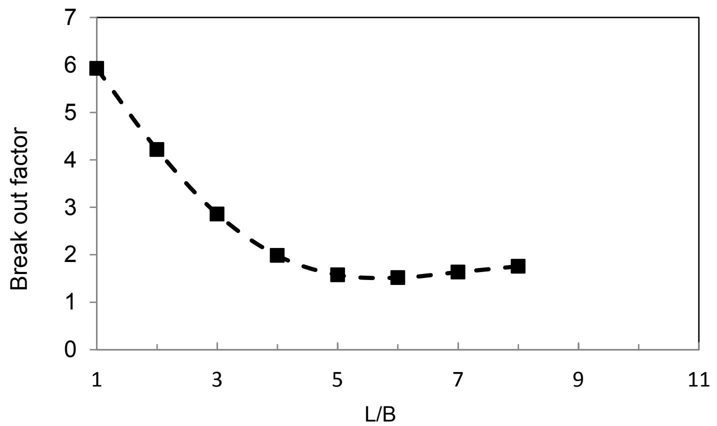

Figure-7: Pullout capacity as a function of L/B ratio (H/B=3 and B=50 mm)

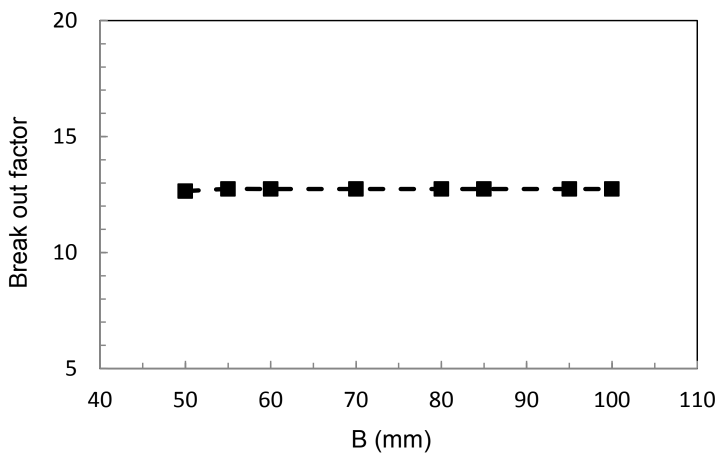

Figure-8: Pullout capacity as a function of size (H/B=3 and L/B=4)

5. CONCLUSIONS

A back-propagation neural network was used to demonstrate the feasibility of ANNs to predict the uplift capacity of anchor foundations in cohesion less soils. A neural Network model is developed for the uplift capacity of anchor foundation. The six input parameters to the neural model are the embedment ratio, aspect ratio, size, inclination, unit weight and denseness of soil. The output of neural network is the uplift capacity of anchor foundation. ANNs have the advantage that once the model is trained, it can be used as an accurate and quick tool for estimating the uplift capacity of anchor foundation without a need to perform any manual work such as using tables or charts. The main shortcomings of ANNs are the lack of theory to help with their development and their limited ability to explain the way they use the available information to arrive at a solution. In addition, like all empirical models, the range of applicability of ANNs is constrained by the data used in the model calibration phase and ANNs should thus be recalibrated as new data become available. However, despite the aforementioned limitations, the results of this study indicate that ANNs have a number of significant benefits that make them a powerful and practical tool for anchor capacity prediction of anchor foundation in cohesionless soils.

REFERENCES

Baker, W., and Kondner, R. (1966). "Pullout load capacity of a circular earth anchor buried in sand.” National Academy of Sciences, Highway Research Record No. 108, 1-10.

BALLA, A. (1961). “The resistance to breaking –out of mushroom foundations for pylons.” 5th International conference on Soil Mechanics and Foundation Engineering, Paris, France, vol. 1, 569-576.

Das,B.M., and Seely,G.R.(1975). “Pullout resistance of vertical anchors.” ASCE Journal of the Geotechnical Engineering Division, 101:87-91.

Dickin, E.A. (1988). “Stress displacement response of buried plates and pipes.” International Conference on Geotechnical Centrifugal Modeling, Paris. A.A. Balkema, Rotterdam, The Netherlands, pp. 205-214

Dickin,E.A. qnd Leung, C.F. (1983). “Centrifugal Model tests on vertical anchor plates.” ASCE Journal of Geotechnical Engineering 109: 1503-1525.

Fausett, L. V. (1994). “Fundamentals neural networks: Architecture, algorithms, and applications.”Prentice-Hall Inc., Englewood Cliffs, New Jersey.

Flood, I., and Kartam, N. (1994). “Neural networks in civil engineering I: Principles and understanding.” *J. Computing in Civil Engrg*, ASCE, 8(2), 131-148.

Frydman. S. and Shaham.I, (1889). “Pullout capacity of slab anchors in sand.”Journal of Canadian Geotechnical Engineering ,26: 385-400.

Gardner, M. W., and Dorling, S. R. (1998). “Artificial neural networks (The multilayer perceptron) - A review of applications in the atmospheric sciences.” *Atmospheric Environment*, 32(14/15), 2627-2636.

Goh, A. T. C. (1995). “Neural networks for evaluating CPT calibration chamber test data.” *Microcomputers in Civil Engineering*, **10** (2), 147–51.

Goh, A. T. C. (1995a). “Back-propagation neural networks for modeling complex systems.” *Artificial Intelligence in Engineering*, 9, 143-151.

Goh, A. T. C. (1995b). “Empirical design in geotechnics using neural networks.”*Geotechnique*, 45(4), 709-714.

Goh, A. T. C. (1995c). “Modeling soil correlations using neural networks.” *J. Computing in Civil Engrg.*, ASCE, 9(4), 275-278.

Goh, A. T. C. (1996a). “Neural-network modeling of CPT seismic liquefaction data.” *J. Geotech. Engrg.*, ASCE, 122(1), 70-73.

Goh, A. T. C. (1996b). “Pile driving records reanalyzed using neural networks.” *J. Geotech. Engrg.*, ASCE, 122(6), 492-495.

Hecht-Nielsen, R. (1990). *Neurocomputing*, Addison-Wesely Publishing Company.

Hornik, K., Stinchcombe, M., and White, H. (1989). ‘‘Multilayer feedforward networks are universal approximators.’’ *Neural Networks,* 2,359–366.

Maier, H. R., and Dandy, G. C. (2000). “Neural networks for the prediction and forecasting of water resources variables: A review of modelling issues and applications.” *Environmental Modelling & Software*, 15(2000), 101-124.

Majer J. (1968). "Zur berechnung von zugfundamenten." *Osterreichische,* Bauzeitschrift, 10(5), 85–90.

Maren, A., Harston, C., and Pap, R. (1990). “Handbook of neural computing applications.” Academic Press, Inc., San Diego, California.

Masters, T. (1993). “Practical neural network recipes in C11.” Academic, San Diego.

Md. Rokonuzzaman and Toshinori Sakai (2013).” Model tests and 3D finite element simulations of uplift resistance of shallow rectangular anchor foundations.” International Journal of Geomechanics, Vol. 12, No. 2, April 1, 2012.

Merifield, R. S., and Sloan, R. S. (2006). "The uplift pullout capacity of anchors in frictional soils." Can. Geotech. J., 43(8), 852-868.

Meyerhof, G. G., and Adams, J. I. (1968). "The ultimate uplift capacity of foundations." Can. Geotech. J., 5(4), 225-244.

Mors H. (1959). "The behaviour of mast foundations subjected to tensile forces." *Bautechnik*, 10, 367-78.

Moselhi, O., Hegazy, T., and Fazio, P. (1992). "Potential applications of neural networks in construction."*Can. J. Civil Engrg*, 19, 521-529.

Muhammad Buhari and Sanusi Sani Adamu(2012). "Short-Term Load Forecasting Using Artificial Neural Network."Proceeding of the International MultiConference of Engineers and Computer Scientists,Volume-I,Hong Kong.

Murray, E.J., and Geddes, J.D. (1987). "Uplift of anchor plates in sand." ASCE Journal of geotechnical engineering ,Volume-5, 202-215.

Najjar, Y. M., and Basheer, I. A. (1996). ''A neural network approach for site characterization and uncertainty prediction.'' Geotechnical Special Publication, 58-1, 134–148.

Ovesen, N.K. (1981). "Centrifugal tests of the uplift capacity of anchors."10th International Conference on soil Mechanics and Foundation Egineering, Stockholm, Sweden, vol. 1. pp. 717-722.

Ripley, B. D. (1996). "Pattern recognition and neural networks." Cambridge University Press.

Rowe, R. K. and Davis, E. H. (1982). "The behavior of anchor plates in sand."*Geotechnique*, 32(1), 25-41

Sakai, T., and Tanaka, T. (2007). "Experimental and numerical study of uplift behavior of shallow circular anchor in two-layered sand." *J. Geotech. Geoenv. Engrg.,* 133(4), 469-477.

Rowe, R.K., and Davis, E.H. (1982). "The behavior of anchor plates in sand." Geotechnique, 32: 25-41.

Rumelhart, D. E., Hinton, G. E., and Williams, R. J. (1986). ''Learning internal representation by error propagation.'' Parallel distributed processing, Vol. 1, Chap. 8, D. E. Rumelhart and J. L. McClelland, eds., MIT Press, Cambridge, Mass.

Shahin, M. A., Jaksa, M. B., and Maier, H. R. (2000). "Predicting the settlement of shallow foundations on cohesionless soils using back-propagation neural networks." *Research Report No. R 167*, The University of Adelaide, Adelaide.

Smith, J.E. (1962). "Deadman anchorage in sand."U.S. Naval civil engineering laboratory, Technical Report R 199

Smith, M. (1993). "Neural networks for statistical modeling." Van Nostrand–Reinhold, New York.

Stone, M. (1974). ''Cross- validatory choice and assessment of statistical predictions.'' *J. R. Stat. Soc.,* B 36, 111–147.

Vermeer, P. A., and Sutjiadi, W. (1985). "The uplift resistance of shallow embedded anchors."Proc. 11th Int. Conf. Soil Mech. Found. Engr., San Francisco, 4, 1635-8.

Vesic, A. S. (1971). "Breakout resistance of objects embedded in ocean bottom." J. Soil Mech. Found. Div., 97(SM9), 1183-1205.

Zurada, J. M. (1992). "Introduction to artificial neural systems."West Publishing Company, St. Paul.

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ANALYSIS OF TRAVEL BEHAVIOR IN KHULNA METROPOLITAN CITY

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ABSTRACT

Urban transportation problem has become one of the main problems faced by cities in developing countries. The rapid growth of motorization and city structure, such as mixed land-use and the increasing rate of urbanization are some causes of this phenomenon. Travel survey identifies the start and end of a trip, the trip purpose and the mode of travel as well as the socio-economic characteristics of the trip makers. Travel characteristics are very important resources that give valuable information about travel behavior of people over a period of time across the population. Household interviews were used through questionnaire survey on demographic and socio-economic characteristics in Khulna Metropolitan City. The results of the analysis show that people with higher income and more automobile availability make more travel than people with low income and less automobile availability. The home-based trips take the largest percentage (50%) of people in the study area. The result also indicates that the shopping trips (15%) contribute higher among different trip purpose. The results also show that about 57% of individuals are between 20-50 years. The analysis of this paper reveals that the travel behavior in the study area is quite similar to the middle sized city in Asian countries.

Keywords: *Khulna Metropolitan City, Household survey, Travel behavior, Socio-economic data*

1. INTRODUCTION

Travel behaviour is complex, not only in terms of its motivations, but also in terms of how it manifests itself (Parsons *et al.*, 2002). People travel because they get benefits from it, or more precisely, they get benefits from the things they do or buy at the end of the trip (Puget Sound Regional Council, 2001). The environment in which transport analysis and infrastructure planning is taking place has changed radically during the last few decades. The urbanization in developing countries is dynamic; the big cities in these countries are reporting sustained pressure due to heavy migration from rural areas and high growth of private mode of transportation along with public transportation services (Domencish and McFadden, 1975).

One of the key issues of travel behavior is travel mode choice decision. More choice plays a vital role in transportation planning and policy making in any city. Past research has clearly shown that individual and household socio-economic characteristics have strong influence on mode choice decision (Miller *et al.*, 2005; Bhat and Sardesai, 2006). They identified that income, gender, vehicle ownership, employment status are the most influencing variables in mode choice decision. Residential location and built environment attributes also play an important role in travel mode choice decisions (Pinjari *et al.*, 2007; Frank *et al.*, 2000). Many urban transportation studies have been done in several metropolitan areas such as in Bangkok, Thailand; Manila, Philippine; Kuala Lumpur, Malaysia and Jakarta, Indonesia. Generally, metropolitan area means city with two million people or over (Itorralba, 1988). Although Bangladesh has seven metropolitan cities, very few transportation studies have been conducted to these cities. In Bangladesh, the middle sized cities (i.e. population between 0.3 – 1.0 million) are the majority but, as yet, a few studies have been conducted and the availability of data is limited. The study area, Khulna Metropolitan City (KMC) is one of the most important middle sized cities in south-west of Bangladesh and very urbanizing area. The traffic load is rapidly increasing day by day due to the urbanization trend and changes in socio-economic level of the people. As such, the travel behavior in KMC is carried out. The aim of this paper is to expose the existing dimensions of travel behavior in terms of socio-economic and travel characteristics information such as trips purpose, trips mode, trips distance, cost of trips, household income, and vehicle ownership in Khulna Metropolitan city.

2. METHODOLOGY

The study area is located in Khulna. Total population of this district is 2.38 million based on the Census 2001(BBS, 2001). Approximately, 1.28 of the city population is residing in the city corporation area. The population density of Khulna Metropolitan city (KMC) is around 16242 persons per km2 as compared to the figure 541 persons per km2 for Khulna district. The KMC with its geographical area of 50.57 km2 comprises of five police stations (Thanas). The transportation system of KMC is dominated mainly by public transportation mode. Public transportation systems in KMC include the buses and intermediate public transport (IPT) modes like baby taxi and Easy bike. Khulna City Corporation has 31 wards which almost covered the study area. As a result, these 31 wards were used as 31 zones with one external zone outside the metropolitan area. A large amount of time, money and man power was required to cover all these 32 zones. Accordingly the study area was divided into three major divisions according to land use pattern. The major divisions were Division-1 (Zone 1 to Zone 15), Division-2 (Zone 16 to Zone 23) and Division-3 (Zone 24 to Zone 31). In this study, two zones from each major division were randomly selected for survey and analysis. The selected zones are shown in Table 1.

Table 1: Selected Zone for Study

Major Division	Selected Zones
Division-1	Zone 4, Zone 5
Division-2	Zone-18, Zone 21
Division-3	Zone 26, Zone 28

The socio-economic data are categorized as the number of population, land use patterns, transportation data such as road network and so on. In this study, the socio-economic and population data were collected from Bangladesh Bureau of Statistics, Regional Office, Khulna, Bangladesh. The survey data were categorized into travel pattern data and travel time data. In this study, data relating to the present pattern of individual movement was collected by a home interview survey (HIS). Data were obtained by visiting the house of every respondent.

3. RESULTS AND DISCUSSION

This section provides a descriptive analysis of socioeconomic characteristics of household and individuals obtained from the sample. A total of 233 households were interviewed, and the questionnaires were distributed among 765 individuals over 7 years old. The respondents consist of 395 (49.7%) male and 390 (50.3%) female. Reported 1556 trips were used in the analysis. Lot of trips were generated and attracted within a zone. The reason to this might be the relative size of the zones .That is because the study area was a medium city, where the distance between different land use facilities were not so far.

In this study, the age structure was grouped into 4 age divisions as less than 7 years old, 7-20 years old, 20-50 years old and greater than 50 years of age bracket. It was assumed that the people between 20-50 years old are active and independent travelers. Meanwhile, people between 7-20 years can be active but have some limitations as they have no fixed income yet. For people less than 7 years or more than 50 years old are considered groups who could not travel independently. This group also may travel less or differently than the working age groups. Figure 1 shows the age structure of the household members obtained from the survey. It is seen that 56.9% of individuals is between 20-50 years of age, the 7-20 age bracket followed with 19.3% share. There are 12.2% and 11.6% share for the young children and the elderly.

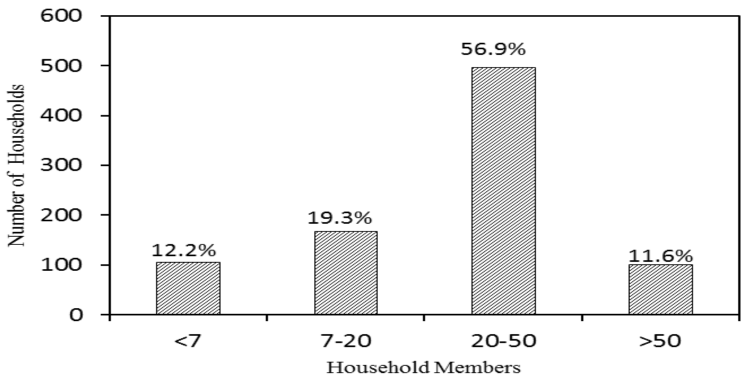

Figure 1: Age Structure of Household Members

The income is one of the most important factors shaping the travel patterns. In this study, the household income was calculated including whole monthly income of the members and head of household. Figure 2 shows the household income structure. It is seen that 89 (38.2%) households have monthly income ranging from BDT10000 to BDT20000. It is also seen that the monthly income ranging from BDT5000 to BDT10000 followed with 80 (34.3%) households and BDT20000 to BDT50000 followed with 38 (16.3%) households.

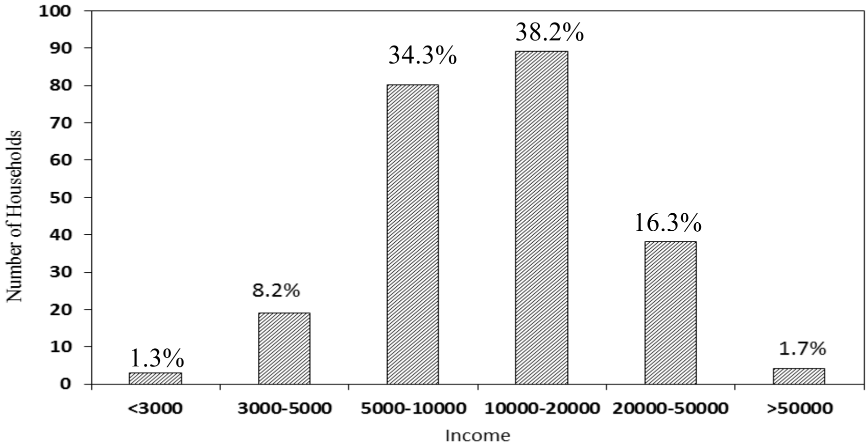

Figure 2: Household Income Structure

Figure 3 shows the total and percentage occupation of respondents. Most of the individuals interviewed came from housewives 242 (31.6%), student 196 (25.4%), businessmen 99 (12.9%), and private service 110 (14.4%). It also shows that the Government employee followed with 44 individuals (5.8%). Meanwhile, retired and others have proportion of 18(2.4%) and 56 (7.3%), respectively.

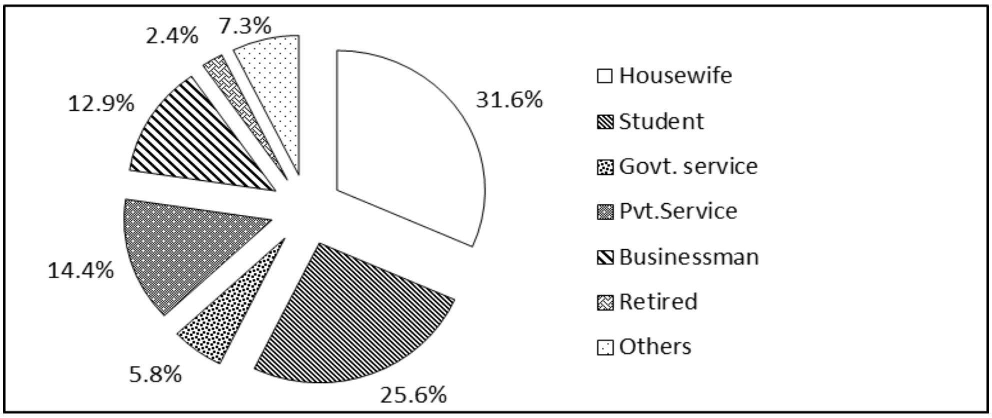

Figure 3: Totals and Percentage Occupation of Respondent

Car occupancy is associated with the access to private vehicles. Table 2 shows the distribution of households belonging to vehicles ownership and vehicle rate per households. It is seen that the car ownership is very low (3.9% share) compare with motorcycle 8.6%. Car ownership includes private and office cars; it is categorized that the role of an office car is similar to private car. More than 65% have no access to private vehicles and about 22% have access to bicycle only.

Table 2: Distribution of Vehicle Ownership

Vehicle ownership	Number	Households (%)	Vehicle Rate Per Household
No Vehicle	153	65.7	-
Bicycle	51	21.9	0.22
Motorcycle	20	8.6	0.09
Car	9	3.9	0.04
Total	233	100.0	-

The distribution of types and numbers of the vehicles are concentrated with the income of the household especially with the upper income group and these groups are much more mobile than lower income groups. Figure 4 shows the relationship between car ownership level and household monthly income. It is seen that the level of car ownership increased rapidly with their income. Higher income group has more luxurious vehicle than the lower income groups.

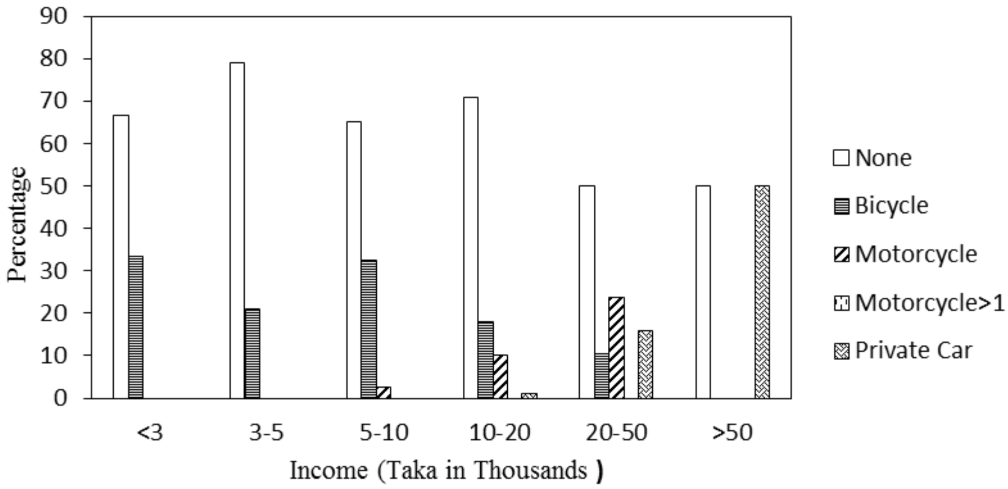

Figure 4: Relationships between Vehicle Ownership and Monthly income

In Khulna, the office hour is different for government and private sector. Government office hour usually finish at 17:00, and different private sectors have different time limit. Figure 5 shows the distribution of trips by time of trip start. It is seen that at 8:00 am to 10:00 am there is peak in the trip distribution.

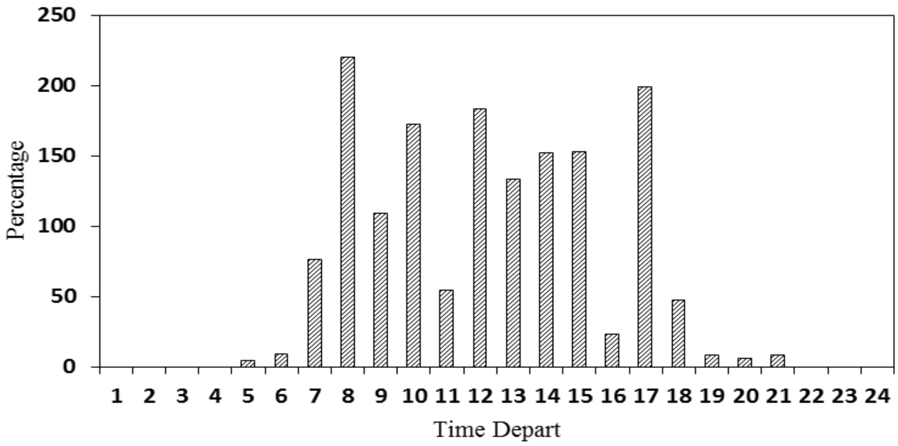

Figure 5: Distribution of Trip by Time of Trip Start

Figure 6 shows that most of the trips are generated from home (50% of the total trips) followed with trips from office (16.6%). Trips generated from education facilities, shopping, recreational and others are 12.5%, 16.2%, 2.4% and 2.3%, respectively.

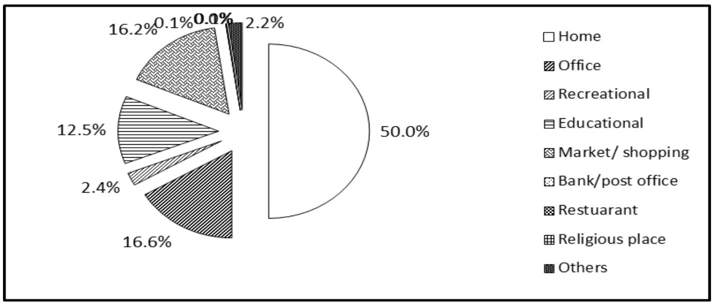

Figure 6: Total trip Based on Facility of Origin

Table 3 shows the daily trips per household for various purposes. It is seen that most of the trip makers are distributed among four purposes; home (50%), work (15%), school (13%) and shopping (12%). Business trip has shared of 8%, recreation 2% and others 1%.

Table 3: Daily Trips per Household for Various Purposes

Trip purpose	No. of Trips	Trips per household per day	Percentage of Trips
Home	780	3.3	50
Work	232	1.0	15
School	195	0.8	13
Shopping	185	0.8	12
Business	119	0.5	8
Recreation	33	0.1	2
Others	12	0.1	1
Total	1556	6.7	100.0

In Khulna, public transport is provided for fixed route which is mainly on the major streets. Many households, especially those who have income at the average level or lower live away from the major streets, walk trips are the first mode to access for the public mode on the major streets. By using cross-tabulation analysis the joint frequencies of the main-mode have been obtained. The number of trips for each mode is presented in Table 4. It is seen that the dominant mode in Khulna is walking with 46.6%. Travel within shorter distance in choice of public modes, favorable mode is rickshaw and auto rickshaw. People choose city bus mainly for long distance trips, also the access to bus is slightly limited as all buses have operated with fixed route as well as fixed stop. There is no doubt that walking is the dominant mode of travel in Khulna. It is important for school, work and shopping purposes, and rickshaw for business, and shopping purposes. Now a day's auto rickshaw has become more popular than rickshaw. For the relatively longer trips for work and business purpose auto rickshaw, bus and motorcycle are frequently used.

Table 4: Numbers of Trips by Mode

Mode	Number	Trips per Household	% of Trips
Walking	725	3.11	46.6
Bicycle	92	0.39	5.9
Rickshaw	351	1.51	22.6
Motorcycle	48	0.21	3.1
Auto rickshaw	218	0.94	14.0
Bus	107	0.46	6.9
Microbus	0	0.00	0
Private car	10	0.04	0.6
Others	5	0.02	0.3
	1556	6.70	100.0

As transportation studies of cities in Bangladesh are very limited, a study of Manado, Indonesia is selected to compare with Khulna Metropolitan City (KMC). Manado is the Capital city of North Sulawesi province in Indonesia. It has played an important role in the entire North Sulawesi as centre of administration, education, trade, business, culture, and tourism. Table 5 shows the comparison of daily trips household for various purposes between KMC and Manado. The result of Manado is based on the result of Rumayar (1992) and the rate of trip generation in Manado was 8.3 trips per household per day; whereas the value for Khulna is 6.7 trips per household per day. Although, the trip per person per day is 1.5 trips compare with 1.8 trips/person/day of

Manado. The rate of trip per person per day can be reflected from several factors such as number active worker, size, and income of a household. It is seen that home-based trip in Khulna is 50% and in Manado 43% share. Whereas, the work trips in KMC and Manado are shared about 15% and 17%, respectively. Educational trip in Khulna (13%) is also lower than Manado (14%). Trips related to shopping in Khulna (about 12%) are found higher than Manado (about 7%).

Table 5: Daily Trips per Household comparison

Trip Purpose	Trips Per Household Per Day		Percentage of Trips	
	KMC	Manado	KMC	Manado
Home	3.3	3.6	50	43
Work	1.0	1.4	15	17
School	0.8	1.1	13	14
Shopping	0.8	0.6	12	7
Business	0.5	1.1	8	13
Recreation	0.1	0.2	2	2
Others	0.1	0.3	1	4
Total	6.7	8.3	100	100

Table 6 shows the comparison of modal split by trip purpose of the study area, KMC and Manado. The access to the private vehicle in KMC is still low with respect to Manado. The average rate is about 4%. Household income is the main factor reflected to this pattern.

Table 6: Mode by Trip Purpose Comparison

City	Mode	Trip purpose (%)				
		Home	Work	School	Shopping	Others
Khulna	Walking	52.8	43.5	50.3	45.9	41.7
	Bicycle	5.9	7.3	7.7	0.5	0.0
	Rickshaw	17.9	34.9	23.1	30.3	25.0
	Motorcycle	3.2	0.4	4.6	0.0	16.7
	Auto Rickshaw	14.0	10.3	7.2	15.7	0.0
	Bus	5.3	2.2	5.6	7.6	16.7
	Microbus	0.0	0.0	0.0	0.0	0.0
	Private Car	0.6	0.4	1.0	0.0	0.0
	Others	0.3	0.9	0.5	0.0	0.0
Manado	Walking	25.08	12.9	25.07	28.0	46.20
	Bicycle	1.20	1.8	.6	1.4	.90
	Motorcycle	15.6	16.3	9.3	10.80	12.30
	Bus	41.1	44.0	57.60	46.30	17.30
	Car	11.4	18.60	5.10	10.80	17.00

4. CONCLUSIONS

This study gives an idea about the travel pattern of the people living in Khulna. The travel pattern generally depends on the age of the traveler, socio-economic conditions, vehicle ownership, etc. It is found that people with higher income generally travels more than people with lower income. Vehicle ownership is directly related to the income of the households. It is also seen that car ownership in Khulna is still low about 3.9% share compare with motorcycle 8.6%. A large proportion of home-based trips imply that most of the trips are limited and have a single purpose. The percentage of home-based trips is 50% of the total. The dominant mode in Khulna is walking with 46.6%. Rickshaw is the second highest mode with 22.6%. Now-a-days auto rickshaw has become more popular about 14% of the total trip's mode. People generally choose walking, rickshaw and auto rickshaw for travelling shorter distances and choose city bus mainly for long distance trips. The analysis of this paper reveals that the travel behavior in KMC is quite similar to the middle sized city in Asian countries.

REFERENCES

Bangladesh Bureau of Statistics (BBS) (2001).

Bhat, C.R. and Sardesai, R. (2006). "The Impact of Stop-Making and Travel Time Reliability on Commute Mode Choice." Transportation Research Part B, Vol. 40, No. 9, pp. 709-730.

Domencish, T.A. and McFadden, D. (1975). "Urban Travel Demand: A Behavioral Analysis." North Holland Publishing Company, Amsterdam, The Netherland.

Frank, L., Bradley, M., Kavage, S., Chapman, J., and Lawton, K. (2008). "Urban form, Travel Time, and Cost Relationships with Tour Complexity and Mode Choice." Transportation, Vol. 35, No. 1, pp. 37-54.

Itorralba, E.C., (1988). "Analysis of Individual Travel Behavior in Bangkok." Master's Thesis, Asian Institute of Technology, Bangkok, Thailand.

Miller, E.J., Roorda. M.J., and Carrasco, J.A. (2005). "A Tour based Model of Travel Mode Choice." Transportation, Vol. 32, pp. 399-422.

Parsons, Brinkerhoff, Quade and Douglas (2002). "Travel Demand model development and application guidelines." State of Oregon: Department of Transportation, USA.

Pinjari, A.R., Pendyala, R.M., Bhat, C.R., and Waddell, P.A. (2007). "Modeling Residential Sorting Effects to Understand the Impact of the Built Environment on Commute Mode Choice." Transportation, Vol. 34, No.5, pp. 557-583.

Puget Sound Regional Council (2001). "Land use and travel demand forecasting models." Review of the literature and operational models, Washington, University of Washington, USA.

Rumayar, A.L.E., (1992). "Analysis of Travel Behavior in Manado, Indonesia". Master's Thesis, Asian Institute of Technology, Bangkok, Thailand.

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COMPUTER AIDED ANALYSIS AND DESIGN OF A COUNTERFORT RETAINING WALL

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ABSTRACT

Retaining wall is a very common structure. But retaining wall designing is a long calculation with lot of trial and error methods. This long calculation is time consuming and also provides possibility of mistakes in calculation which can change the whole design. In this situation computer aided programming is very much helpful and reliable. Based on these necessities a software has been developed which will be very helpful for computer aided analysis and design of counterfort retaining wall. Previously in KUET two projects were taken to develop software for computer aided analysis and design of retaining wall. The first one "Computer Aided Analysis and Design of a Cantilever Retaining Wall" was in 1996 by Lokman Hossain and Aminul Islam from Department of Civil Engineering, KUET. And the second one "Computer Aided Analysis and Design of a Counterfort Retaining Wall" was in 1997 by Md. Shahjahan Ali and Farhana Jesmin from Department of Civil Engineering, KUET. To overcome these shortcomings, in this project a software has been developed in Visual Basic .net language. This software has Graphical User Interface (GUI) which makes it user friendly. It also provides reinforcement detailing diagram which fulfill the final need of designing.

1. INTRODUCTION

Retaining walls are structures constructed for the purpose of retaining earth or other materials like coal, ore, water etc. Provision of retaining walls become necessary in the construction of hill roads, embankment, bridge abutments, basement in buildings, water reservoirs, in preventive measures against soil erosion, in lands copping etc.

In the days of 90's in KUET two projects were taken to develop software for computer aided analysis and design of retaining wall. The first one "Computer Aided Analysis and Design of a Cantilever Retaining Wall" was in 1996 by Lokman Hossain and Aminul Islam from Department of Civil Engineering, KUET. And the second one "Computer Aided Analysis and Design of a Counterfort Retaining Wall" was in 1997 by Md. Shahjahan Ali and Farhana Jesmin from Department of Civil Engineering, KUET. Both of those softwares were programmed in FORTRAN language which were very challenging because programming like these big and critical require patience and excellency. Unfortunately FORTRAN has no Graphical User Interface (GUI) facility for developed software, it was not possible to provide GUI in those softwares. As a result both softwares only deliver results without any reinforcement detailing diagram.

2. OBJECTIVES

The objectives of the project are as follows:

(i) Development of a general Visual Basic .net program for the analysis of counterfort retaining wall for any height of wall.

(ii) Comparison of results obtained in computer analysis to that by conventional method (i.e. by hand calculation).

3. METHODOLOGY

3.1 Determination of The Earth Pressure

It is necessary to determine the pressure exerted be the soil in designing a retaining wall. The pressure mainly depends upon the type of backfill material predominantly used in calculating earth pressure.

As per Rankine's theory, the intensity of active earth pressure per unit vertical area of the wall is given by the relation

$$Pa = \gamma h \frac{1 - sin\varphi}{1 + sin\varphi}$$

Where, Pa = The intensity of active earth pressure
γ = Unit weight of soil
φ = Angle of internal friction of soil

3.2 Condition of Stability of Retaining Wall

To avoid failure of the retaining wall it is necessary that the requirements are satisfied:
(i) It should not overturn.
(ii) It should not slide.
(iii) The maximum pressure at the toe should not exceed the safe bearing capacity of soil.

(i) Check against overturning

$$F.S. = \frac{M_R}{M_o} > 2$$

Where, M_R = Sum of overturning moment about toe
M_o = Sum of resisting moment about toe
F.S. = Factor of safety

(ii) Check against sliding

$$F.S. = \frac{F_R}{F_S} > 1.5$$

Where, F_S = Total horizontal force tending to slide the wall
F_R = Total resisting force = $\mu \times \sum W$
μ = Coefficient of friction between base concrete and soil
$\sum W$ = Sum of the vertical loads

(iii) Check against maximum pressure at toe

$$P_1 = \frac{\sum W}{b(1 + \frac{6e}{b})} < P_o$$

$$P_2 = \frac{\sum W}{b(1 - \frac{6e}{b})} < P_o$$

$$e < \frac{b}{6}$$

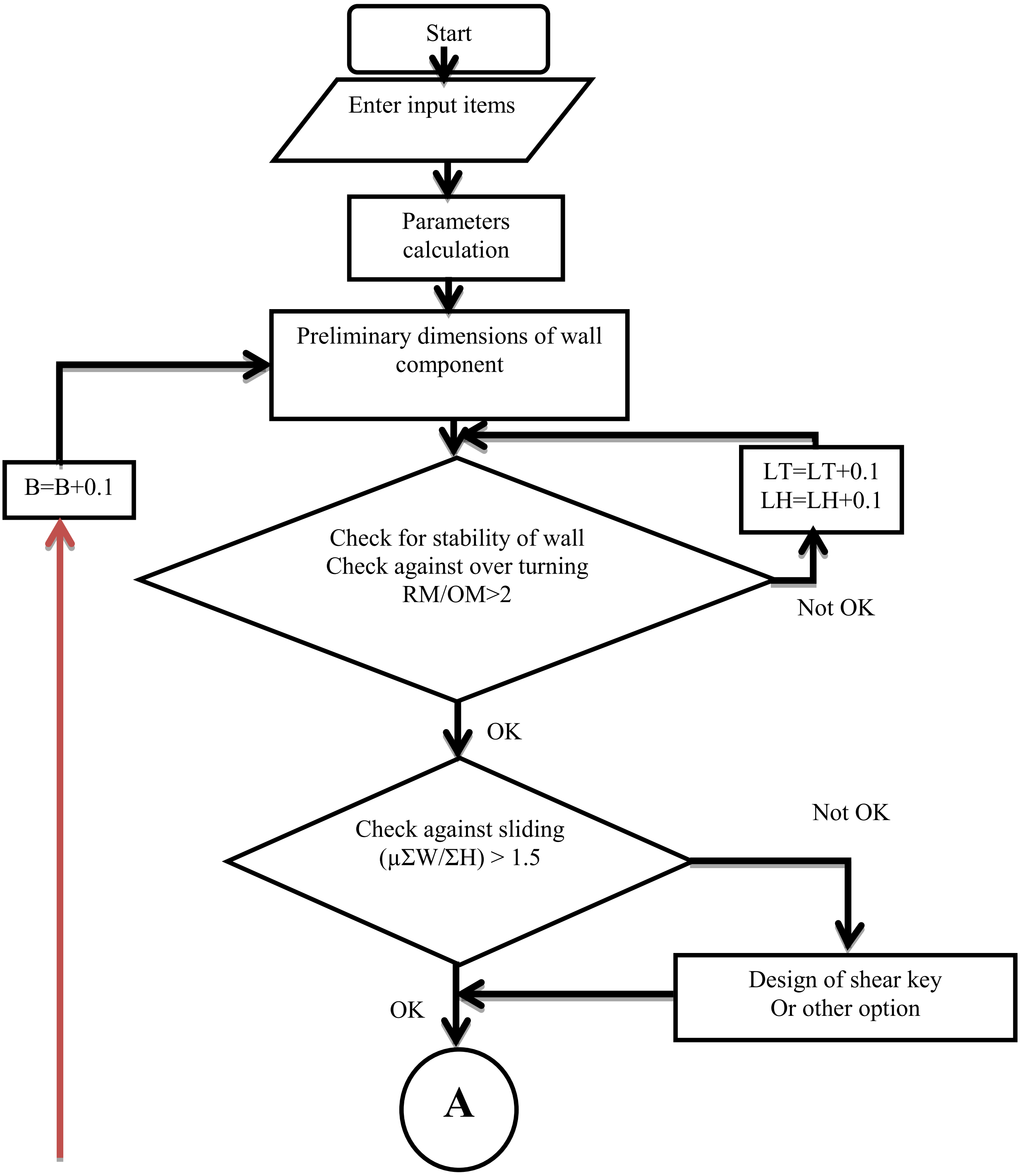
Where, P_1 = Intensity of soil pressure at toe
P_2 = Intensity of soil pressure at heel
e = Eccentricity
b = Width of base
P_O = Safe bearing capacity of soil
$\sum W$ = Sum of the vertical loads

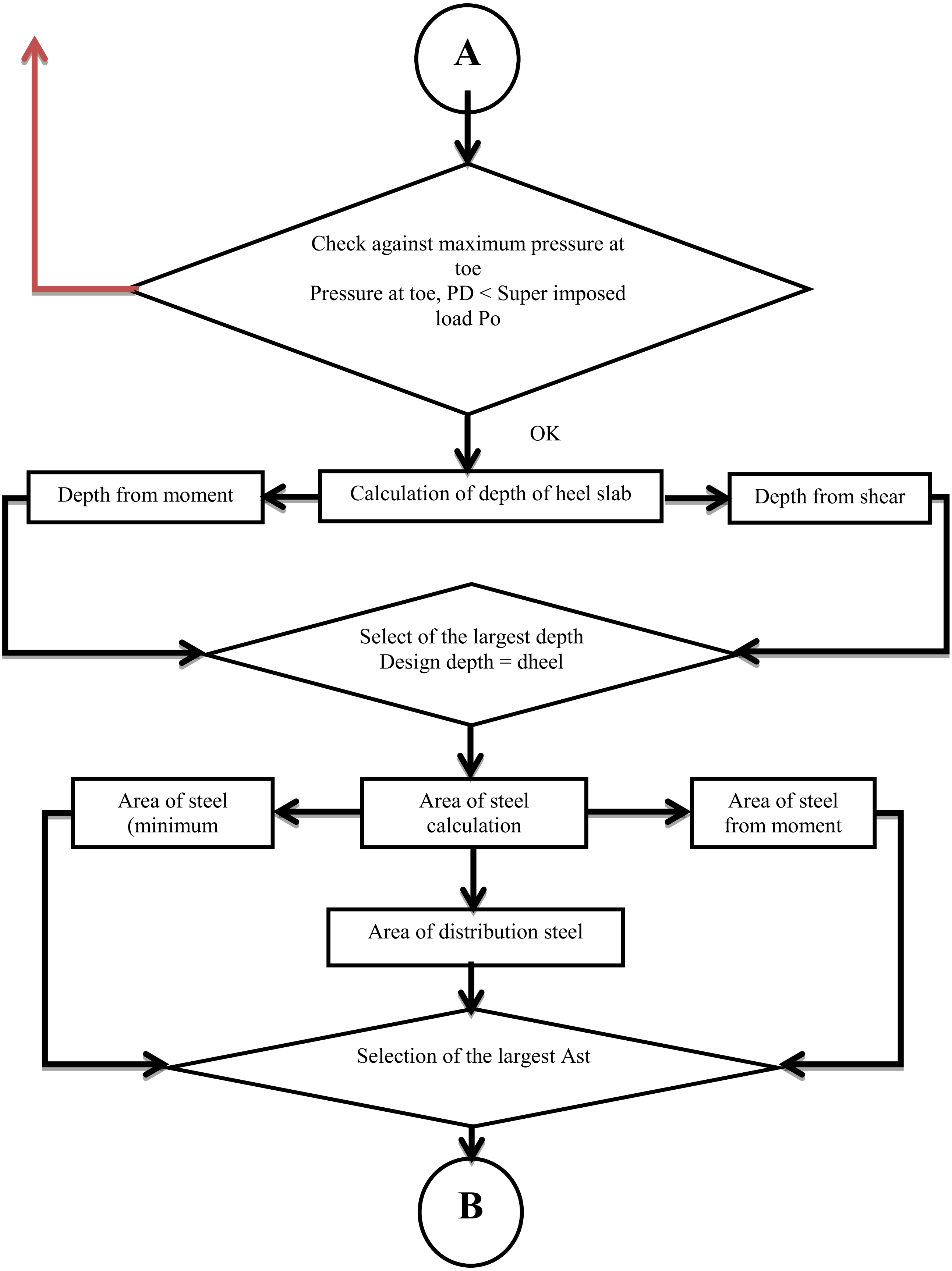
3.3 Steps to be Followed in The Design

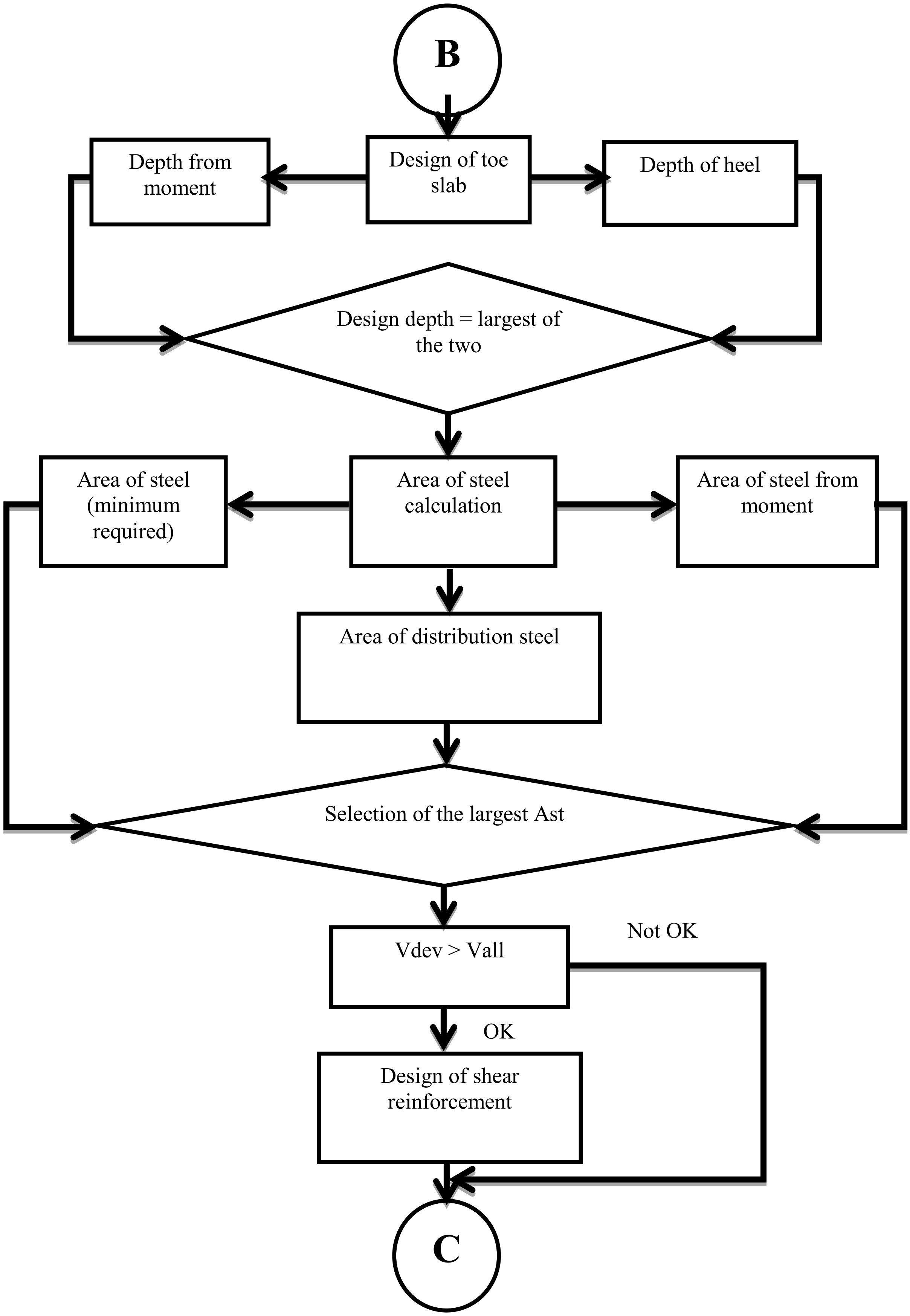
(1) Calculation of earth pressure by Rankine's formula.
(2) Determination of preliminary dimensions of the wall components.
(3) Checking the stability of the wall against
(i) Overturning
(ii) Sliding
(iii) Soil pressure at toe

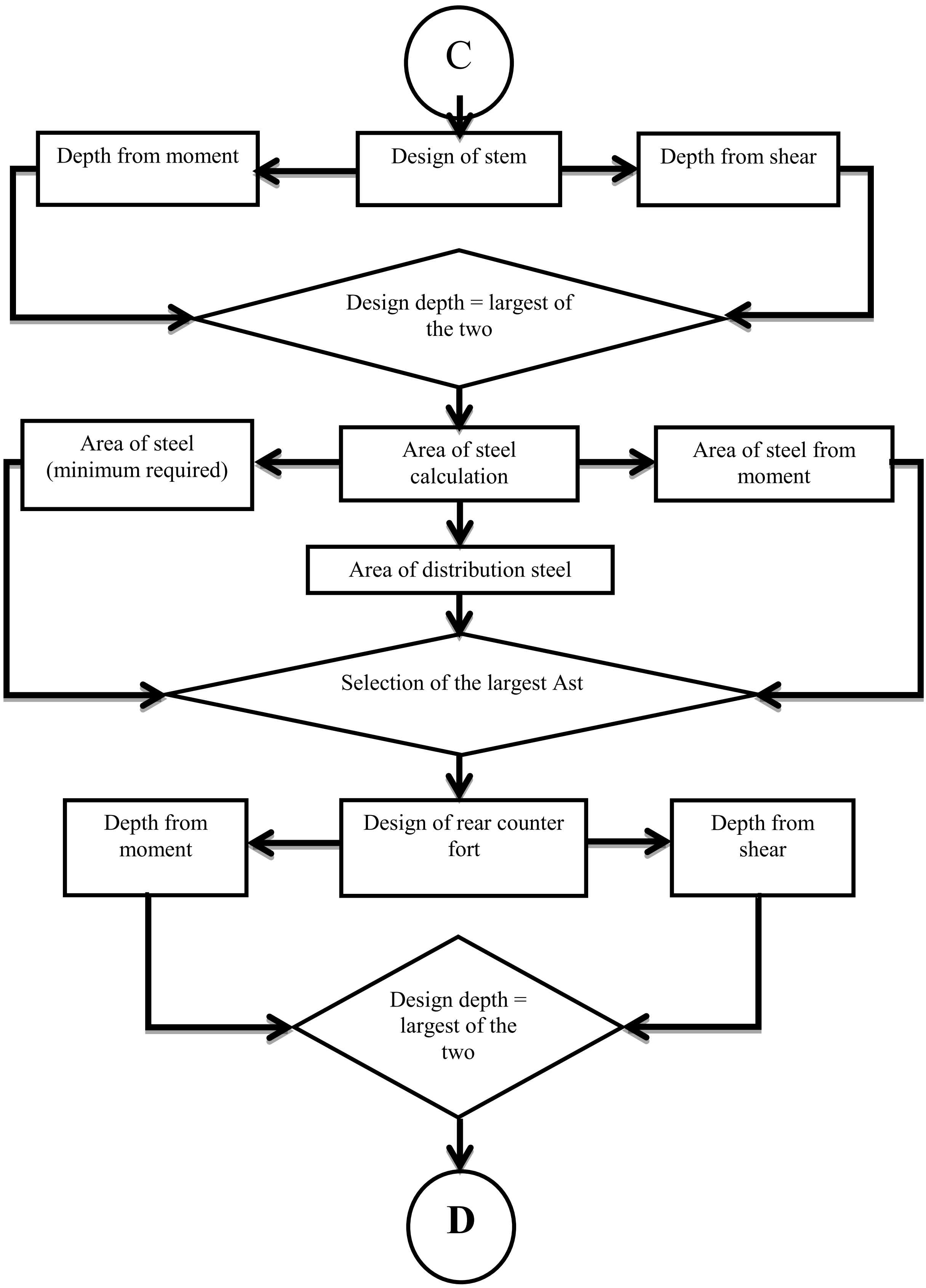
(4) Design of heel slab
(5) Design of toe slab
(6) Design of stem
(7) Design of rear counterfort
(8) Design of front counterfort
(9) Design of key

3.4 Flow Chart

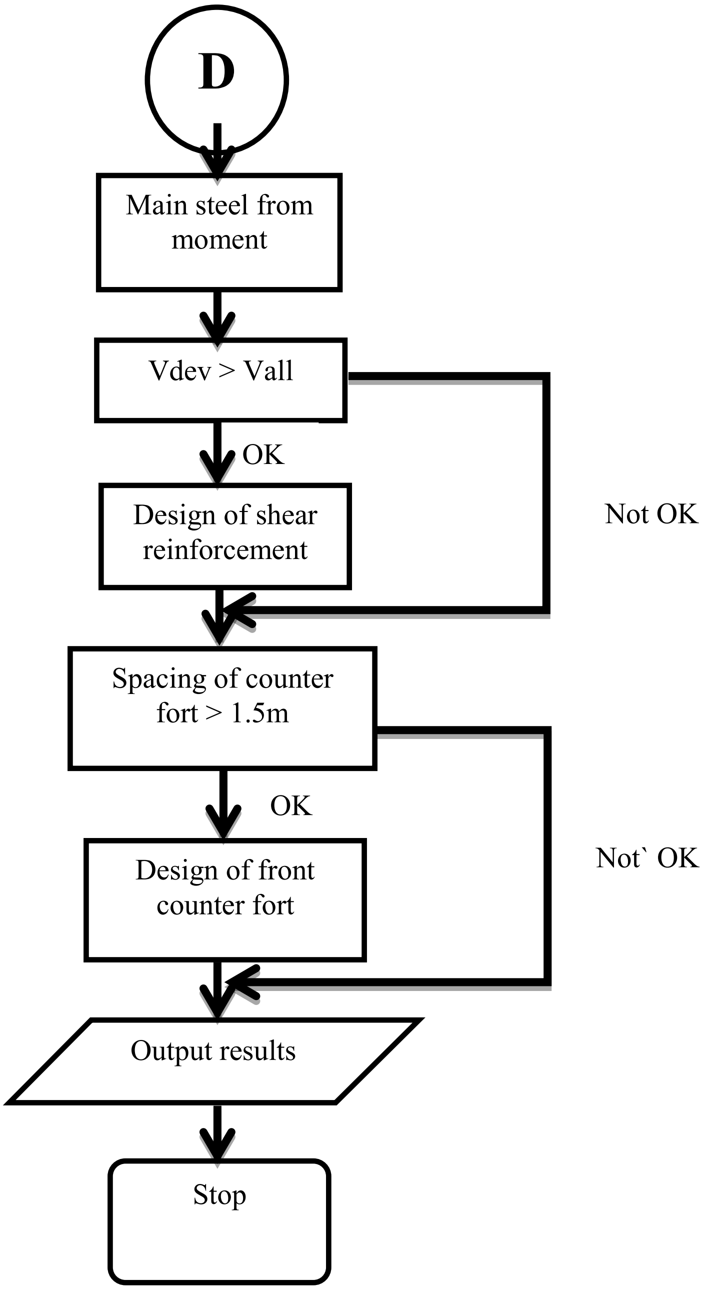



A
Check against maximum pressure at toe
Pressure at toe, PD < Super imposed load Po
OK
Depth from moment
Calculation of depth of heel slab
Depth from shear
Select of the largest depth
Design depth = dheel
Area of steel (minimum
Area of steel calculation
Area of steel from moment
Area of distribution steel
Selection of the largest Ast
B


B
Depth from moment
Design of toe slab
Depth of heel
Design depth = largest of the two
Area of steel (minimum required)
Area of steel calculation
Area of steel from moment
Area of distribution steel
Selection of the largest Ast
Not OK
Vdev > Vall
OK
Design of shear reinforcement
C


C
Depth from moment
Design of stem
Depth from shear
Design depth = largest of the two
Area of steel (minimum required)
Area of steel calculation
Area of steel from moment
Area of distribution steel
Selection of the largest Ast
Depth from moment
Design of rear counter fort
Depth from shear
Design depth = largest of the two
D

4.


D
Main steel from moment
Vdev > Vall
OK
Design of shear reinforcement
Not OK
Spacing of counter fort > 1.5m
OK
Design of front counter fort
Not` OK
Output results
Stop

4. RESULTS AND DISCUSSIONS

Table for comparison of results from software and hand calculation

Data name	Result from software	Result from hand calculation
Design parameters		
f_c	5	5
f_s	140	140
n	18.67	18.67
K	0.4	0.4
J	0.87	0.87
R	0.87	0.87
K_a	0.35	0.35
K_p	2.82	2.86
Preliminary dimensions of wall components		
Height of wall, h (m)	8.5	8.5
Base width, b (m)	5.25	5.25
Toe projection, L_t	2.1	2.1
Clear spacing of counterfort, L (m)	2.75	2.75
Thickness of stem at bottom (m)	0.71	0.71
Thickness of stem at top (m)	0.3	0.3
Thickness of base slab (m)	0.71	0.71
Heel projection (m)	2.44	2.44
Check for stability of wall		
Horizontal earth pressure on the wall tending to overturn, P_H (N)	271498.36	269211
Overturning moment, M_O (Nm)	469914.75	466314
Stabilizing moment, M_R (Nm)	1968147.46	1967911
Factor of safety against overturning	4.19	4.22
Check against sliding		
Factor of safety against sliding	1.01	1.11
Check against maximum pressure at toe		
Net moment about D (Nm)	1498232.7	1501597
Eccentricity, e	-0.26	-0.29
P_{min} at toe (N/m^2)	69710.31	65746
P_{max} at heel (N/m^2)	128217.79	130931
Data name	Result from software	Result from hand calculation
P_B (N/m^2)	93113.3	91820
P_G (N/m^2)	101007.17	100635.5
Design of heel slab		
Net downward load at E, (N/m^2)	42607.89	39931

Data name	Result from software	Result from hand calculation
Net downward load at G, (N/m^2)	62216.16	62629.5
Maximum –ve B.M. (Nm)	26851.85	25165
Depth from moment, d (mm)	176.02	170
Allowable T_C	0.32	0.32
Maximum S.F. (N)	58585.85	54905
Depth from S.F., d (mm)	182.09	172
Final effective depth, d (mm)	652.33	654
Reinforcement near F		
At support (mm^2)	12 mm dia bars @ 100 mm c/c	12 mm dia bars @ 100 mm c/c
At mid span (mm^2)	12 mm dia bars @ 100 mm c/c	12 mm dia bars @ 100 mm c/c
Reinforcement near G		
At support (mm^2)	12 mm dia bars @ 100 mm c/c	12 mm dia bars @ 100 mm c/c
At mid span (mm^2)	12 mm dia bars @ 100 mm c/c	12 mm dia bars @ 100 mm c/c
Distribution reinforcement of heel slab	10 mm dia bars @ 100 mm c/c	10 mm dia bars @ 100 mm c/c
Design of toe slab		
Net upward load at C, (N/m^2)	52001.98	47996
Net upward load at B, (N/m^2)	75404.97	74070
Maximum B.M. (Nm)	32772.08	30245
Depth from moment, d (mm)	194.46	186
Maximum S.F. (N)	71502.72	65995
Depth from S.F., d (mm)	222.24	206
Final effective depth, d (mm)	652.33	654
Reinforcement near C		
At support (mm^2)	16 mm dia bars @ 150 mm c/c	16 mm dia bars @ 150 mm c/c
At mid span (mm^2)	16 mm dia bars @ 150 mm c/c	16 mm dia bars @ 150 mm c/c

Data name	Result from software	Result from hand calculation
Reinforcement near B		
At support (mm^2)	16 mm dia bars @ 150 mm c/c	16 mm dia bars @ 150 mm c/c
At mid span (mm^2)	16 mm dia bars @ 150 mm c/c	16 mm dia bars @ 150 mm c/c
Distribution reinforcement of heel slab	10 mm dia bars @ 100 mm c/c	10 mm dia bars @ 100 mm c/c
Check for shear		
T_C	0.1	0.11
Design of stem		
Intensity of earth pressure at G (N/m^2)	71353.46	71000
B.M. (Nm)	44967.55	44745
Depth from moment, d (mm)	227.78	227
Maximum S.F. (N)	98111.01	97625
Depth from S.F. d (mm)	304.94	305
At support (mm^2)	12 mm dia bars @ 50 mm c/c	12 mm dia bars @ 50 mm c/c
At mid span (mm^2)	12 mm dia bars @ 100 mm c/c	12 mm dia bars @ 100 mm c/c
Distribution reinforcement of stem	10 mm dia bars @ 250 mm c/c	12 mm dia bars @ 100 mm c/c

Data name	Result from software	Result from hand calculation
Design of rear counterfort		
Main reinforcement	11 nos. 22 mm dia bars	11 nos. 22 mm dia bars
Horizontal tie	2 legged 10 mm dia bar @ 100 mm c/c	2 legged 10 mm dia bar @ 100 mm c/c
Vertical tie	2 legged 10 mm dia bar @ 100 mm c/c	2 legged 10 mm dia bar @ 100 mm c/c
Design of rear counterfort		
Main reinforcement	8 nos. 22 mm dia bars in 2 layers	8 nos. 22 mm dia bars in 2 layers
Vertical tie	2 legged 12 mm dia bar @ 950 mm c/c	2 legged 10 mm dia bar @ 1400 mm c/c
Design of key		
Depth of key (m)	4.6	4.6
Width of key (m)	0.71	0.71
Extra weight (N)	518679.13	518710
Active earth pressure (N)	365463.59	367260
Total weight (N)	1074865.41	1034987

(i) For the design of counterfort type retaining wall, the following information were obtained:
- Dimension and
- The reinforcement of the different parts of retaining wall.

(ii) Using software program, retaining wall of any height can be economically designed within a short period of time. The design engineers will be universally benefited by using this computer program.

(iii) The design and analysis of an 8.5 m height retaining wall was done by conventional method and it is compared with the results obtained by the developed Visual Basic .net program.

(iv) The design and analysis of a 7.5 m height retaining wall by conventional method from the previous second project was compared with the results obtained by the developed Visual Basic .net program.

5. CONCLUSIONS

For the analysis and design of a counterfort retaining wall, computer program has been developed. The same retaining wall is analyzed and designed by hand calculation for checking the results obtained by the computer programming. It is found that the results obtained by hand calculation are almost equal to those obtained by the computer program. So it can be concluded that the developed Visual Basic .net program can be used successfully for the purpose of analysis and design of any counterfort retaining wall of any reasonable height.

REFERENCES

Ali, M. S. (1997) “Computer Aided Analysis & Design Of A Counterfort Retaining Wall”, Project report, Department of Civil Engineering, KUET, Khulna.

Hossain, L. and Islam, A. (1996) “Computer Aided Analysis & Design Of A Cantilever Retaining Wall”, Project report, Department of Civil Engineering, KUET, Khulna.

Kumar, S. (1990) “Treasure of R.C.C. Design”, Standard book house, Delhi.

Nilson, W. (1981) “Design of Concrete Structure”, 7th & 9th edition, McGrawhill book company, Singapore.
Vazirani, V. N. (1987) “Concrete Structures”, 13th edition, Khanna Publishers, New Dehli.
Zak, D.(2010) “Clearly Visual Basic, Programming with MicrosoftR Visual BasicR 2010”, 2nd edition, Course Technology, 20 Channel Center Street, Boston, MA 02210, US.

CHLORIDE ABSORPTION POTENTIALITY OF RESIN MIXED CEMENT CONCRETE EXPOSED TO MARINE ENVIRONMENT

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ABSTRACT

Chloride-induced corrosion is the major deterioration phenomena of Reinforced Concrete (RC) structures. Especially in marine atmosphere, chloride ingress into RC structures and severely damages the structures and warrant untimely expensive repair as well. Rresearchers are approaching with innovative ideas to mitigate this but radical results from construction material side not appeared yet. The objective of this study is to assess the chloride absorption potentiality of resin mixed cement concrete exposed to marine atmosphere. This paper will examine, how ion exchange resin combining with cement mortar, could be a potentially effective option to reduce chloride induced corrosion of RC structures. A number of immersion tests were conducted using mortar specimens of High-Early Strength Portland Cement (HESPC) and Polymer Cement (PC). The mortar specimens contained 0%, 1%, 2% and 3 % IER by volume. The prepared specimens were immersed into 10% sodium chloride (NaCl) solution and chloride contents at different depths in all specimens were measured for different time regime. The test results signify the potential enhancement of chloride absorption and binding into the concrete by resin admixture and further this was confirmed and validated by Electron Probe Micro Analysis (EPMA). This potential uses of IER into cement matrix could be a good option for real RC structures for mitigating the chloride induced corrosion.

Keywords: corrosion, absorption, resin, potential, environment

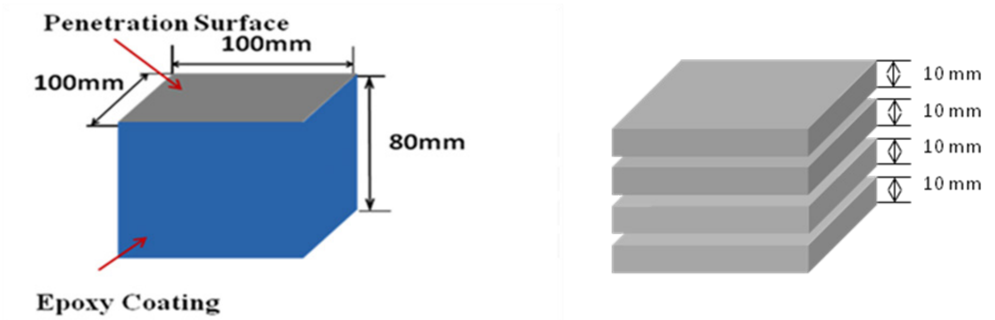
1. INTRODUCTION

Concrete is the most versatile and widely used construction material all over the world but corrosion, especially chloride induced corrosion which may lead to reduction in strength, serviceability, and aesthetics of the RC structures is probably the most vital and common threat for material durability. As a result, durability of RC structures in chloride laden environment is of great interest to design engineers, infrastructure owners and researchers (Shi et al. 2012). In case of RC structures, with passing of time, chlorides from the environment penetrate into the structures and get accumulated, what we call the matured concrete structures. The accumulation and ingress of chloride ion corrode the reinforcement (Elsener 2002). Due to chloride induced corrosion, structures require immediate repair and untimely maintenance. If we could identify the phenomena of chloride ingress into the concrete structures and the chemistry how it accelerates the corrosion, then the development of countermeasure would be the effective solution in controlling the chloride attack. The repair of damaged RC structures can be done, using some ready-mixed mortars (Batis et al. 2003), prepared by using different kinds of additives, such as ion exchange resin, silica fume or fly ashes. These admixture substances should have good characteristics quality compatible with concrete and should not alter the properties of concrete as well. In the present study, a typically available commercial Ion (anion) Exchange Resin (IER) has been examined with two types of cement to justify and determine the chloride absorption and binding potentiality of IER. IER are polymers of spherical beads of 0.5 to 1.0 mm diameter and insoluble substances, containing loosely held ions which are able to be exchanged with other ions. These exchanges take place without any physical alteration to the ion exchange material. This results in the reduction of free chloride in the pore solution and increase the alkalinity of concrete structures which ultimately re-passivate the steel reinforcement. A number of immersion tests were conducted using small mortar specimens of High-Early Strength Portland Cement (HESPC) and Polymer Cement (PC) which contains 0%, 1%, 2% and 3 % IER by volume. The resin absorbs and binds chloride ions from the matured concrete structures and liberates hydroxyl ions into the pore solution and enhances the alkalinity. This paper will examine, how ion exchange resin combining with cement mortar, could be a potentially effective option to bind chloride and reduce chloride induced corrosion of RC structures.

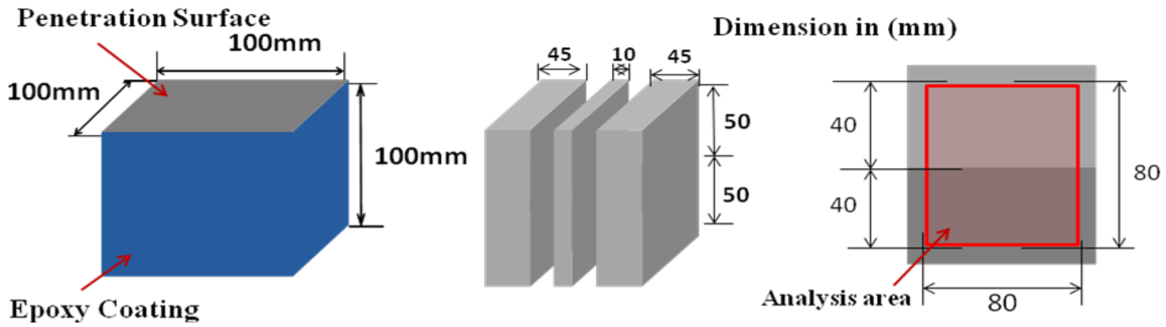
2. METHODOLOGY OF STUDY

2.1 Materials and mix proportions

Immersion tests were conducted to examine the chloride absorption and binding capacity of IER. Two types of specimens were prepared both using HESPC and PC, one type for potentiometric titration analysis and the other one for Electron Probe Micro Analysis (EPMA), whose geometry and dimensions are shown in Figure1. A strongly basic & typical commercially available anion exchange resin has been used in this study. The total exchange capacity of resin is greater than 1.25eq/L (Cl^- form) and the moisture holding capacity is 49 to 55% with 660 g/L-R apparent density. The physical properties and chemical compositions of HESPC and PC are listed in Table 1. The specimens and their mix proportions are listed in Table 2 & Table 3, where 0% IER mixed specimens were the control specimens and 1%, 2% & 3% IER mixed specimen were the test specimens.



(a) Main Specimen for Titration analysis (b) Sliced Specimen for Titration Analysis



(c) Main Specimen for EPMA (d) Sliced Specimen for EPMA (e) EPMA Analysis area

Figure 14: Geometry and dimensions of specimens for titration analysis in (a) & (b) & for EPMA in (c), (d) & (e)

Table 1: Physical properties and chemical compositions of HESPC and PC

Cement Type	Density (g/cm^3)	Chemical Composition (% by mass)							
HESPC	3.14	**(Na_2O+K_2O)**	**SiO_2**	**Al_2O_3**	**Fe_2O_3**	**CaO**	**MgO**	**SO_3**	**NaCl**
		1.624	20	5.01	2.79	65.9	1.55	3.12	0.006
PC	3.15	**Latex fibre**	**(SiO_2 +NaCl)**		**Al_2O_3**	**Hydraulic cement (OPC)**			
		12-23	40-50		6.00	20-25			

Table 2: Test specimens and their mix proportions for HESPC (kg/m^3)

Table 3: Test specimens and their mix proportions for PC (kg/m^3)

Specimen Type	IER (%)	W/ C Ratio	Water	Cement	Resin	Remarks
S1: IER (0%)	0	0.5	315	1750	0	Control Specimen
S2: IER (1%)	1	0.5	315	1724	12	Test Specimen
S3: IER (2%)	2	0.5	315	1698	23	Test Specimen
S4: IER (3%)	3	0.5	315	1673	35	Test Specimen

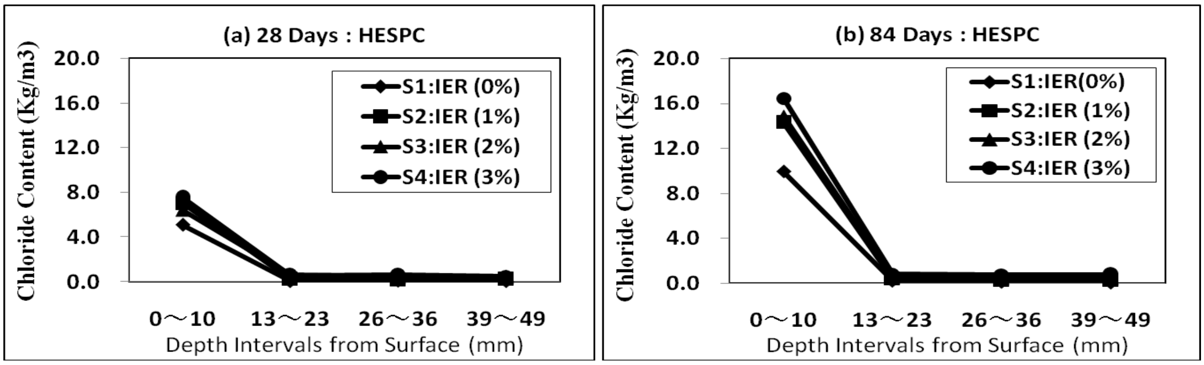
2.2 Exposure Conditions and Analysis of Test Specimens

After casting, each specimen cured by wrapping in wet cloth film to avoid leaching out of chloride ions from the cement matrix. This curing procedure was applied to the specimen for a period of 14 days. At the end of curing period, five sides of each specimen were sealed by epoxy paint (see Figure1(a) & Figure1(c)) so that the chloride penetration can occur only in one-direction. Immediately after sealing, all cubes were immersed into chloride penetration regime, consisting of 1 day of ponding in 10% sodium chloride solution followed by 6 days of drying with constant temperature of 20°C and 60% RH. Specimens were exposed to the above mentioned penetration regime for a period of 28 days, 56 days and 84 days. For titration analysis, after each exposure regime, specimens were sliced into the layers from 0 to 10 mm, 13 to 23 mm, 26 to 36 mm & 39 to 49 mm as shown in Figure 1(b) and grounded separately for potentiometric titration analysis against $AgNO_3$. For EPMA, specimens were sliced into layers and analyzed as shown in Figure 1(d) &Figure 1(e).

3. RESULTS AND DISCUSSIONS

3.1 Titration Analysis

The trend of chloride accumulation and absorption in HESPC and PC mortar specimens at different depths for different time periods were measured quantitatively by silver nitrate potentiometric titration method and presented in Figure 2 & Figure 3. The presence of a large amount of chlorides at the surface layer (0 to 10 mm)



Specimen Type	IER (%)	W/ C Ratio	Water	Cement	Sand	Resin	Remarks
S1: IER (0%)	0	0.5	293	586	1259	0	Control Specimen
S2: IER (1%)	1	0.5	293	586	1248	12	Test Specimen
S3: IER (2%)	2	0.5	293	586	1238	23	Test Specimen
S4: IER (3%)	3	0.5	293	586	1227	35	Test Specimen

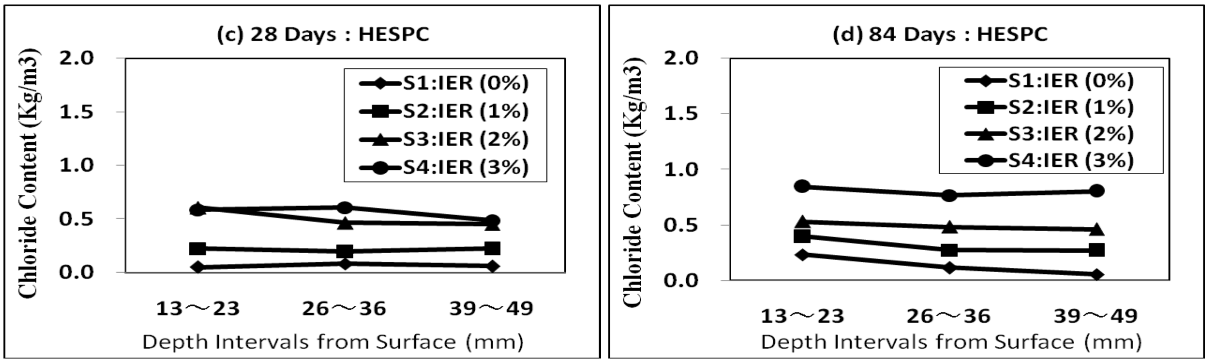


Figure 2: Chloride profile from surface in (a) 28 days: HESPC & (b) 84 days: HESPC specimen and Chloride profile except surface layer in (c) 28 days: HESPC & (d) 84 days: HESPC specimen.

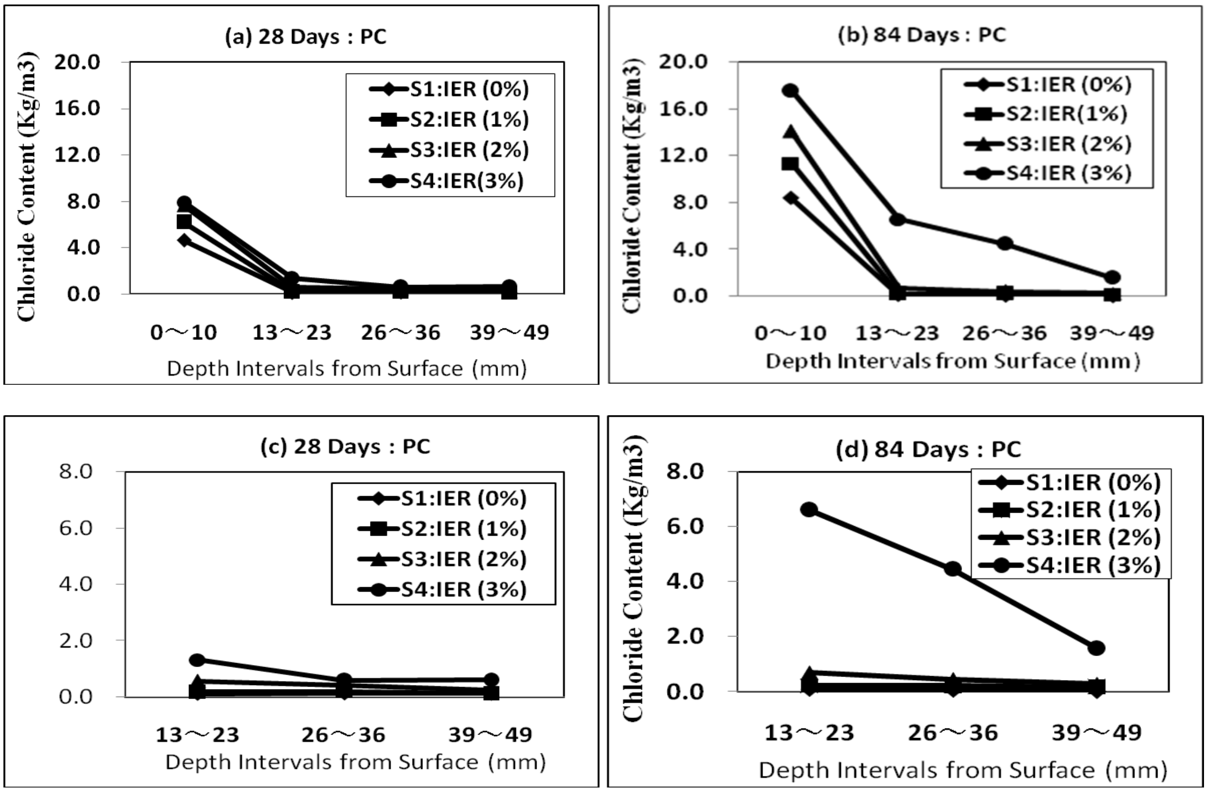


Figure 3: Chloride profile from surface in (a) 28 days: PC & (b) 84 days: PC specimen and Chloride profile except surface layer in (c) 28 days: PC & (d) 84 days: PC specimen.

was due to its close proximity to the ponding surface. There are also other possibilities of salt precipitation and accumulation in the surface layer due to some defects such as blow holes, voids and cracks during the drying periods. In general, the chlorides inside the concrete structures mainly participate in corrosion process and corrode the reinforcement. As free chlorides are main contributor in reinforcement corrosion, the surface layer (0-10 mm) was ignored. The chloride content at inner layers for HESPC presented in Figure 2(c) & Figure 2(d) and for PC presented in Figure 3(c) & Figure 3(d). It was observed that, total chloride content in all depths increased with increment of IER (admixture) percentage in all specimens. It was also observed that, the amount of total chloride content consistently decreased with increase of distance from the surface of the specimens. Test specimens S4 which contained 3% IER (admixture) showed the largest amount of chloride absorption in all depths whereas the control specimens S1 showed the least amount of chloride absorption at every depth. Thus, it clearly represents that, there exists a direct relationship between admixture percentage and increment of chloride content in concrete specimens. Generally, the summation of bound chloride and free chloride content equals to the total chloride content. Hence, the total chloride, bound chloride and free chloride content for different time exposure for HESPC and PC was calculated by using this phenomenon and presented in Figure 4 & Figure 5. The results revealed that test specimen showed much better performance of chloride binding than control

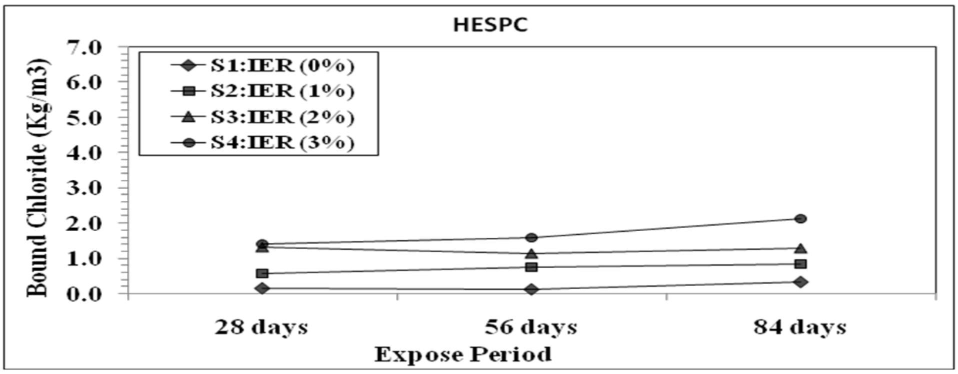


Figure 4: Bound chloride content change with exposure period in HESPC specimens.

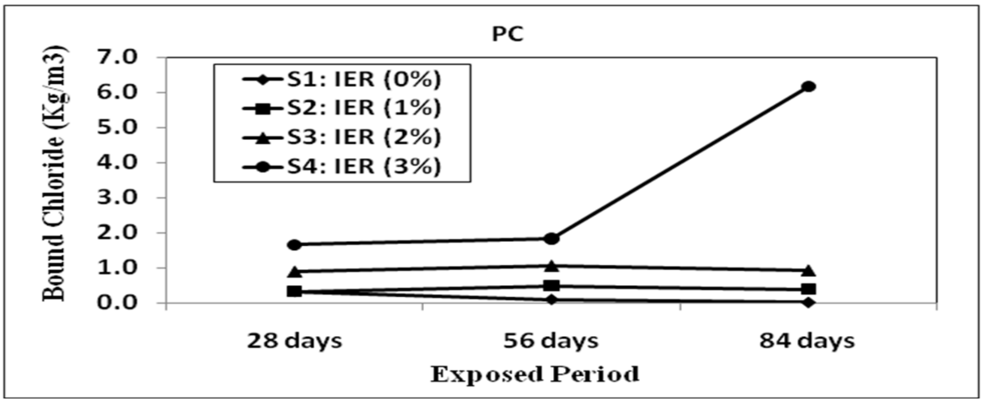


Figure 5: Bound chloride content change with exposure period in PC specimens.

specimens for all time periods as shown in Figure 4 and Figure 5. Hence, this phenomenon was the indication that, if cement matrix contains resin, then with the passes of time, more amount of penetrated chloride will be bound with the resin beads leaving less amount of available free chloride to corrode the reinforcement. In this chloride binding, newly mixed IER admixture plays a dominant role. However, these results confirmed the increment of chloride binding with exposure period and IER was the dominant ingredient at this chloride binding. Bound chloride content was increased with increase of IER percentage and 3% ion exchange resin mixed specimens showed highest amount of bound chloride content in all tested specimens, compared with control and other specimen. This phenomenon was observed both in HESPC and PC. Thus, these results proved the significant usage of admixture for increasing chloride binding in concrete. Once chloride ions penetrate into the mortar specimens and contact with the resin, they effectively absorb chloride ions, exchanging hydroxyl ions into the pore solution and fix them into their beads consistently. It results in reducing free chloride ions in the pore solution and hence decreasing the chloride induced deterioration. It also increases the alkalinity of the cement matrix and ultimately reduces the corrosion probability of concrete structures.

3.2 EPMA

Total chloride ion distribution in the cement mortar specimens were also obtained through EPMA technique. The EPMA results of ion distribution after 84 days for both HESPC and PC were shown in Figure 6 & Figure 7 respectively. The EPMA was done only for the specimens containing 0% and 3% IER both for HESPC and PC. The EPMA results revealed that, due to the containment of IER in the cement mortar, the accumulation of much amount of chloride has been observed in test specimens than control specimens. The chloride level scale showed higher concentration of chloride ions in case of 3% IER mixed specimen than 0% IER mixed specimen. The images clearly depicted the chloride absorption and binding phenomena due to the existence of resin admixture

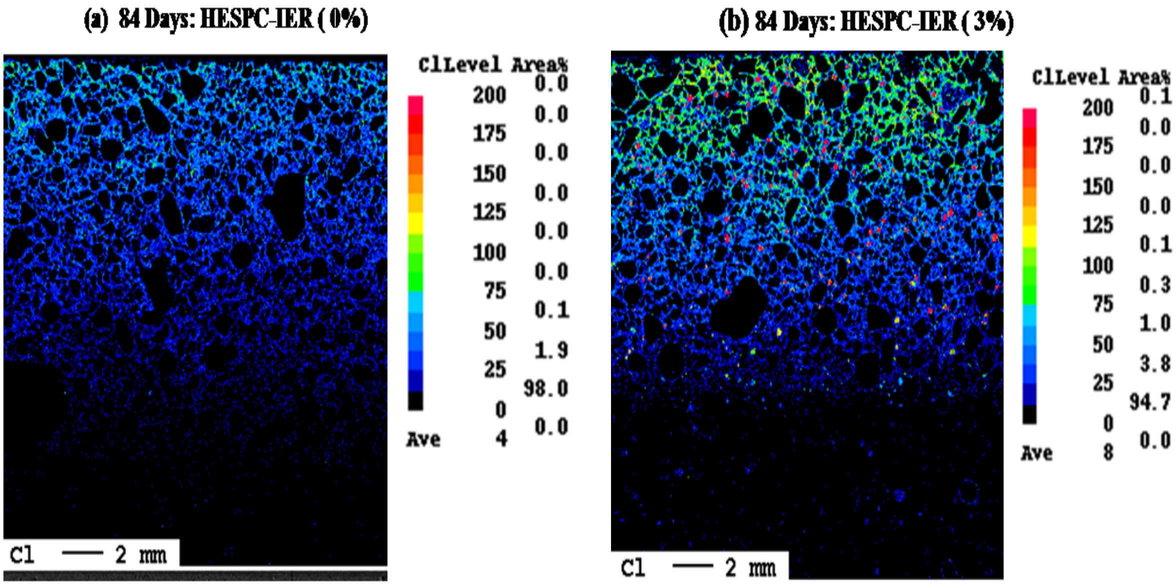


Figure 6: Chloride ion distributions after 84 days for HESPC in (a) & (b)

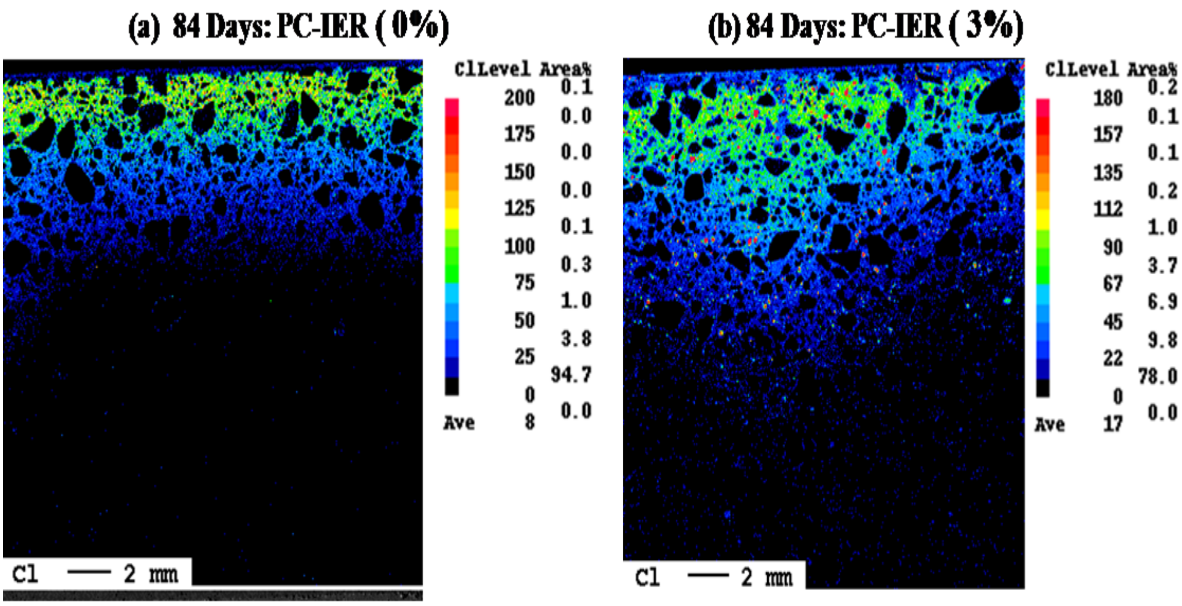


Figure 7: Chloride ion distributions after 84 days for PC in (a) & (b)

in mortar. As already discussed that IER has a wonderful capacity of absorbing and binding of chloride ions when comes in contact with them. In this case, chloride ions penetrated into mortar specimens by diffusion process and absorbed into the resin beads. In case of control specimen although it did not contain resin admixture also showed a little amount of chloride accumulation and absorption. This accumulation was possible due to the penetration of chloride inside the specimen just for concentration and velocity gradient. This EPMA showed the phenomena of much amount of chloride accumulation inside the specimen due to the existence of resin into the mortar speciemn which was also observed in potentiometric titration analysis. This phenomenon was observed both in HESPC and PC So the EPMA also confirmed and validated the same phenomena of chloride movement and absorption by IER as observed in potentiometric titration analysis.

4. CONCLUSIONS

According to the analysis and results obtained by potentiometric titration method and EPMA , some major conclusions can be derived: Firstly, in case of titration analysis, it was revealed that accumulation and absorption of chloride ions inside the mortar specimen was a direct function of resin (admixture) contents, i.e, much the resin contents, much the chloride absprption and binding. PC showed a higher level of chloride absorption than HESPC in case of potentiometric titration analysis. This might be due to the fact that PC contains a few percentage of latex fibres which could show a better level of chloride absorption and binding. The inconsistency which was observed in the results also might be due to improper curing, moisture condition, w/c ratio, delay of cement hydration and slight variations in exposure conditions. Secondly, the experimental results of the chloride content at different depths were validated by EPMA analysis and the same phenomena was observed and confirmed by EPMA. And finally, it can be concluded that, 84 days is not so much time to understand the phenomena at all. So further investigations with much effective observation and time trend might give a more realistic and rational result. As it was observed that existence of resin in mortar specimen could absorp and bind chloride inside the structures, it could ultimately reduce the chloride concentration at reinforcement area. So this Ion Exchange Resin admixture might be potentially and effectively useful in reducing and restraining chloride induced corrosion probability in RC structures in coastal areas or in marine environmrnt. This technique could ultimately enhance the durability and service life of RC structures in marine environment.

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REFERENCES

Shi at al. (2012). Durability of steel reinforced concrete in chloride environments: An overview. Journal on Construction and Building Materials 30(2012), pp.125-138.

Elsener B (2002). Macrocell corrosion of steel in concrete-Implications for corrosion monitoring. Cement and Concrete Composites 24, pp. 65-72..

Batis at al. (2003). Effects of migrating inhibitors on corrosion of reinforcing steel covered with repair mortar. Cement and Concrete Composites 25, pp.109-115.

JCI (2010). Japan Concrete Institute: Diagnostic Technology of Concrete.

Mori at al. (2006). Applications of Electron Probe Microanalyzer for measurement of Cl Concentration Profile in Concrete. Journal of Advanced Concrete Technology, Vol.4, No.3, pp.369-383.

Neville (1995). Chloride attack of reinforced concrete: an overview. Materials and Structures 28, pp. 63-70.

STUDY ON THE SALINITY AND pH AND ITS EFFECT ON GEOTECHNICAL PROPERTIES OF SOIL IN SOUTH-WEST REGION OF BANGLADESH

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ABSTRACT

The high salinity and pH may lead the changing of geotechnical properties of soils. The south-west region of Bangladesh is selected for the study of salinity and acidity and its effect on geotechnical properties. Samples have been collected from 22 different AILA affected locations of 8 districts (Khulna, Jhalkathi, Satkhira, Bagerhat, Borguna, Vola, Pirojpur and Potuakhali). The physical properties, strength and compressibility of the collected samples are investigated. The variation of different properties of soil with salinity and pH has been evaluated. Specific trend in variation is obtained for some properties which attracts the geotechnical engineers to consider these issues in their design.

Keywords: ***Salinity, pH, Engineering properties, Physical properties, Soil salinity.***

1. INTRODUCTION

Salinity and pH are common parameters used to characterize pore fluids of all geo-materials. Salinity is the amount of dissolved salt in pore fluid. Saline soil is a non-alkali soil containing soluble salts, mostly sodium chloride (NaCl). Other salts such as magnesium chloride (MgCl2), potassium chloride (KCl), gypsum (CaSO4.2H2O), sodium sulphate (NaSO4.2H2O) and magnesium sulphate (MgSO4) may also be present. (Salman et al. 2011). After cyclonic storm AILA ripped the south-west part of Bangladesh one year ago, thousands of acres of land are turning into a vast wasteland due to increasing salinity in these areas. Saline soils may have some unfavourable properties such as high compressibility, low bearing capacity and swelling capability (Chittaranjanet al. 2011). Addition of salt solution sharply increases the undrained shear resistance of soil (Naeini et al. 2011).The soil pH is a measure of the acidity or basicity in soils. PH leads to changes in anion and cation exchange capacity of soil to a small extent (Umesha et al. 2012).pH of soil can also be defined by acid rain and growing industrialization in this region. Acid rain results in changes in physico-chemical characteristics of soil due to cation exchange (Sharma et al. 2011). The high salinity in the soils of this region may have been consequently changing geotechnical properties of soils from the past. The soil salinity and pH in the south west region of Bangladesh are growing day by day. So it is necessary to study the salinity and pH effect on soil properties. The main objectives of this paper are to evaluate the variation and relationship of different properties of soil with salinity and pH of soil in south west region of Bangladesh.

2. METHODOLOGY

2.1 Collection of Soil Samples

Soil samples were collected from 22 different locations of south-west region of Bangladesh. The soil samples were collected from different regions of Khulna, Satkhira, Borguna, Jhalkathi, Pirojpur, Bagerhat,Bhola and Potuakhali.Then different tests were performed to determine the physical properties, engineering properties, salinity and pH of collected soils samples and the variation of soil properties with salinity and pH of soil were represented graphically.

Soil samples were collected from the following locations :

Khulna-

- Koyra
- Khalishpur
- Nurnagar
- Jaynagar
- Mailmari
- Dakop
- Botiaghata
- Shipyard
- Sonadanga
- Paikgaccha

Jhalkathi-

- Hetalbunia

Pirojpur-

- Hajigonj

Borguna-

- Amtoli

Bagerhat-

- Bagerhat Sadar
- Chitalmari

Bhola-

- Bhola Sadar
- Charpation

Satkhira-

- Shyamnagar
- Debhata
- Assasuni

Potuakhali-

- Baufol
- Dasmina

These areas are roughly marked by red indications in the following map as shown in Figure-1:

Figure-1: Locations of soil sample collection indicated in Bangladesh map

The following tests were done to determine the physical and engineering properties, salinity and pH of collected soil samples.

- Specific Gravity (ASTM D854)
- Atterberg Limit (ASTM D 4318)
- Grain Size Analysis (ASTM D 422)
- Moisture Content (ASTM D 2216-90)

- Unit Weight (ASTM D 1556-00)
- Shear Strength (ASTM D 2166)
- Consolidation (ASTM D 4186)
- Salinity (Titration – BS 1993)
- pH (ASTM D 4972-01)

3. RESULTS AND DISCUSSION

3.1 Soil Classification

From the Casagrande Plasticity Chart (Figure-2 and Table-1), it is found that 21 soil samples are in same category. They are in MI or OI group (Inorganic silts of medium compressibility and organic silts). Only one sample is in the MH or OH group (Inorganic silts of high compressibility and organic clays).

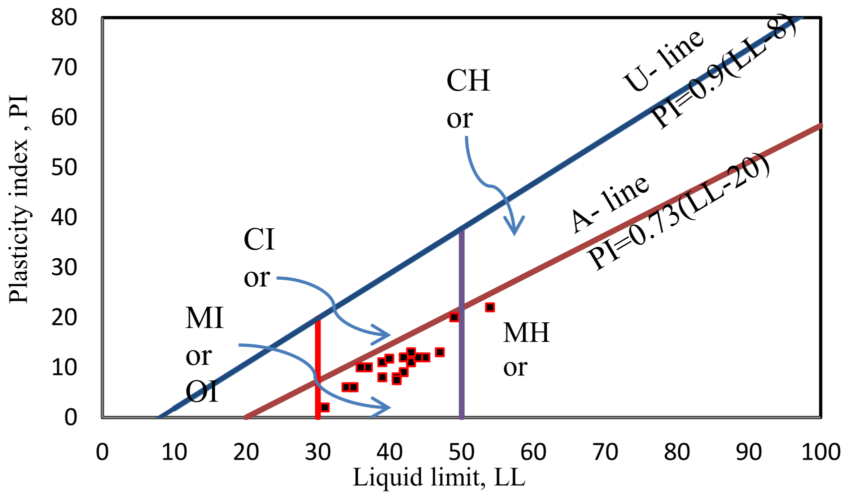


Figure -2: Casagrande Plasticity Chart

Table -1: Soil Classification according to Casagrande Plasticity Chart

Major divisions	Group symbols	Typical name
Silts & clays (liquid limit less than 50)	MI	Inorganic silts of medium compressibility
	CI	Inorganic clays of medium plasticity
	OI	Organic silts , organic silty clays (low plasticity)
Silts & clays (liquid limit greater than 50)	MH	Inorganic silts of high plasticity
	CH	Inorganic clays of high plasticity
	OH	Organic clays (medium to high plasticity),organic silts

3.2 Variation of Physical Properties, Strength and Compressibility Characteristics of soil with Salinity and pH

3.2.1 Variation of Specific gravity with Salinity and pH

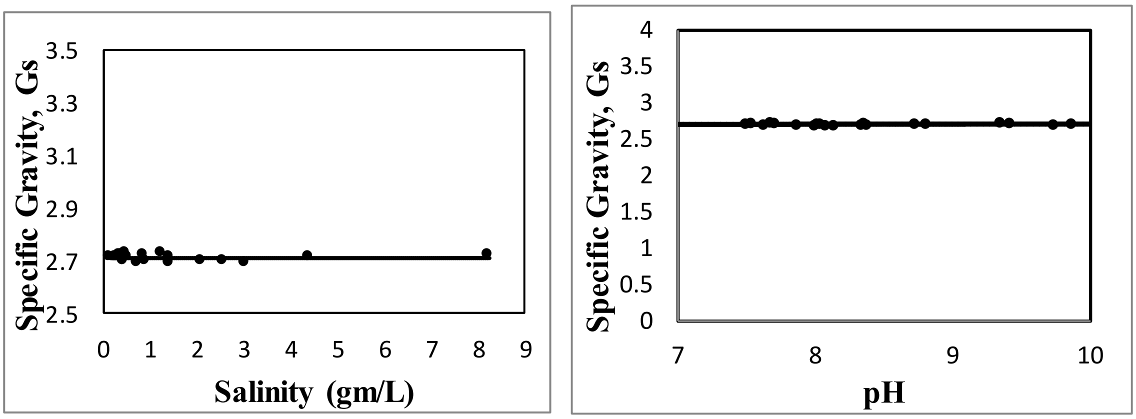


Figure-3: Effect of Salinity and pH on Specific gravity

From Figure-3, it is observed that there is no/small variation of specific gravity with salinity and pH of soil. So it can be concluded that salinity and pH have no significant effect on specific gravity of soil.

3.2.2 Variation of Plastic Limit with Salinity and pH

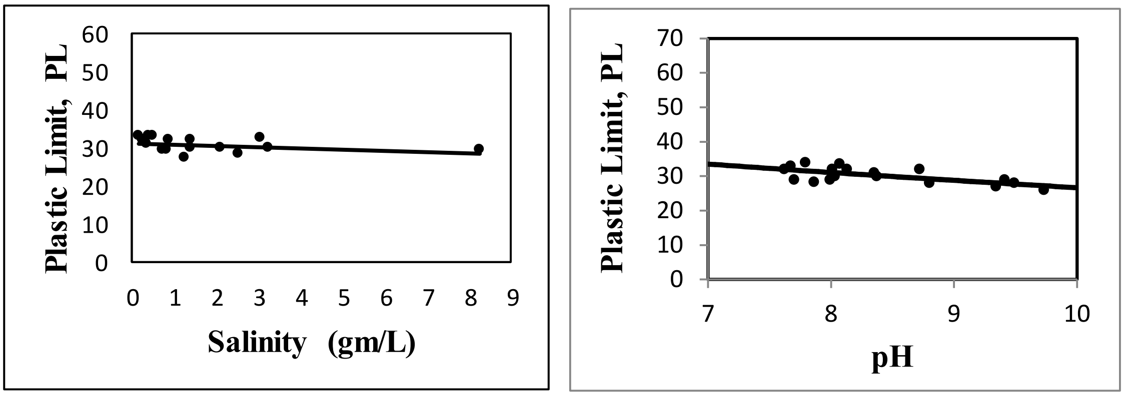


Figure-4: Effect of Salinity and pH on Plastic Limit

Figure-4 shows that plastic limit slightly decreases with increase of salinity. But plastic limit gradually decreases with increase of pH.

3.2.3 Variation of Moisture Content with Salinity and pH

The variation of moisture Content with Salinity and pH has been shown in the following Figure-5. It can be claimed that the moisture content decreases with the increasing of salinity and pH. Also the variation line of moisture content with pH is a convex shaped line. The moisture content increases with a peak when pH is near about 8.5 and after the peak it decreases.

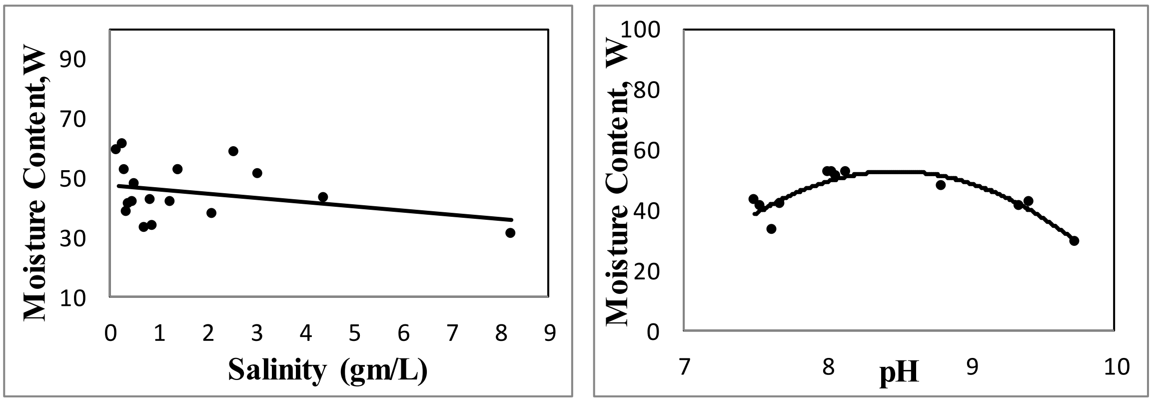


Figure-5: Effect of Salinity and pH on Moisture Content

3.2.4 Variation of Liquid Limit with Salinity and pH

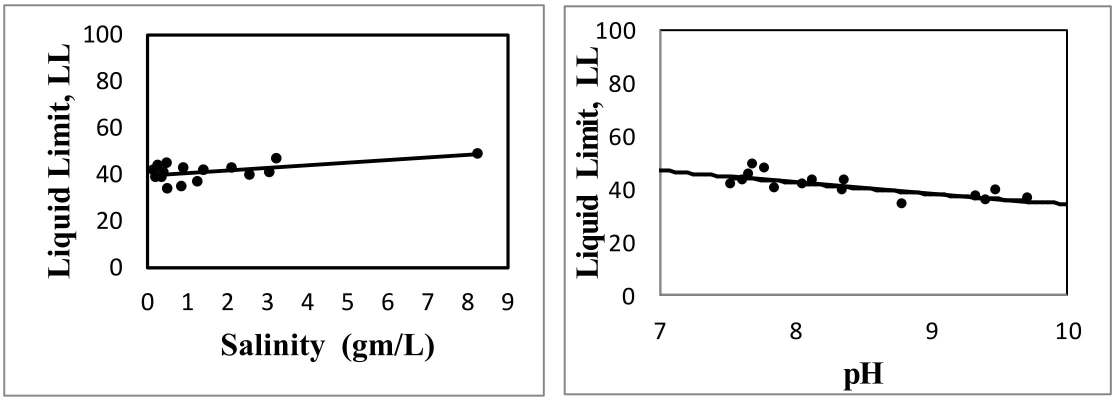


Figure-6: Effect of Salinity and pH on Liquid Limit

From Figure-6, it can be concluded that the liquid limit increases with increase of salinity but liquid limit decreases with increase of pH.

3.2.5 Variation of Shrinkage Limit with Salinity and pH

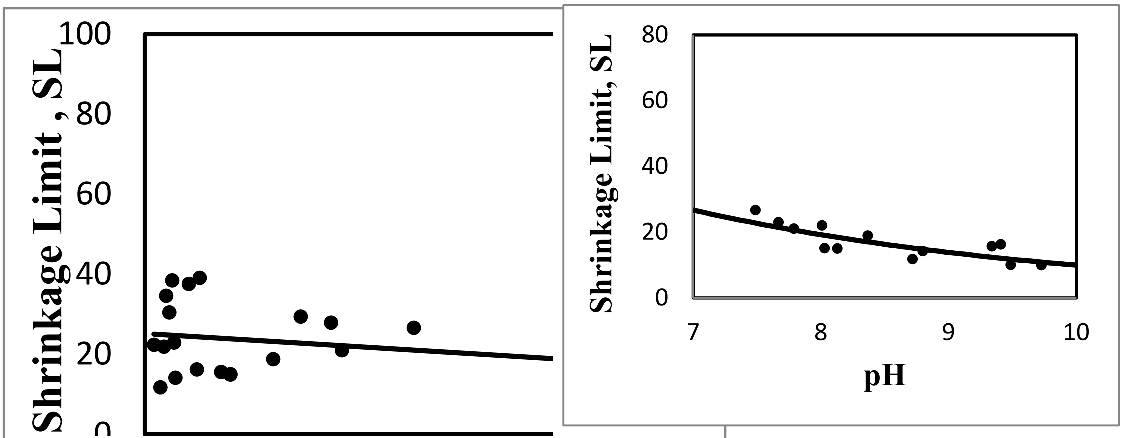


Figure-7: Effect of Salinity and pH on Shrinkage Limit

From Figure-7, it is found that the shrinkage limit decreases with increase of salinity but shrinkage limit decreases with increase of pH.

3.2.6 Variation of Unit weight with Salinity and pH

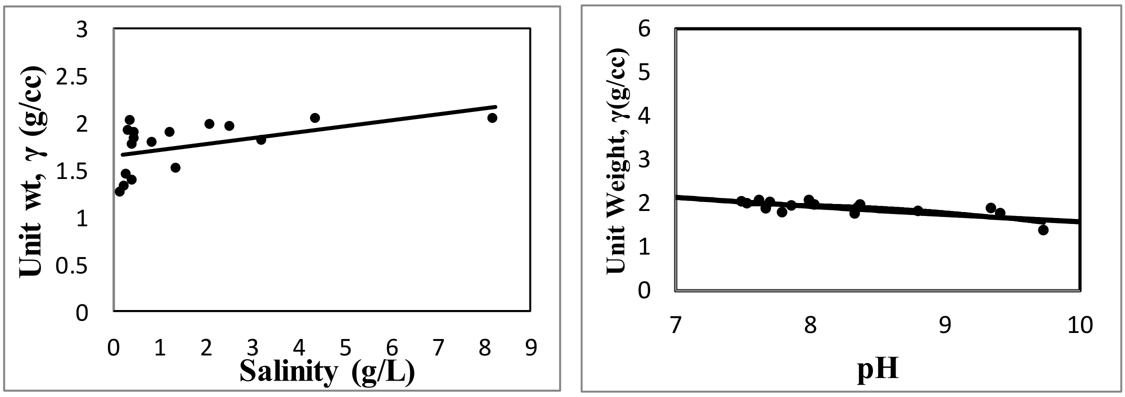


Figure-8: Effect of Salinity and pH on Unit weight

Figure-8 shows that the unit weight increases with increase of salinity but slightly decreases with increase of pH.

3.2.7 Variation of shear strength with Salinity and pH

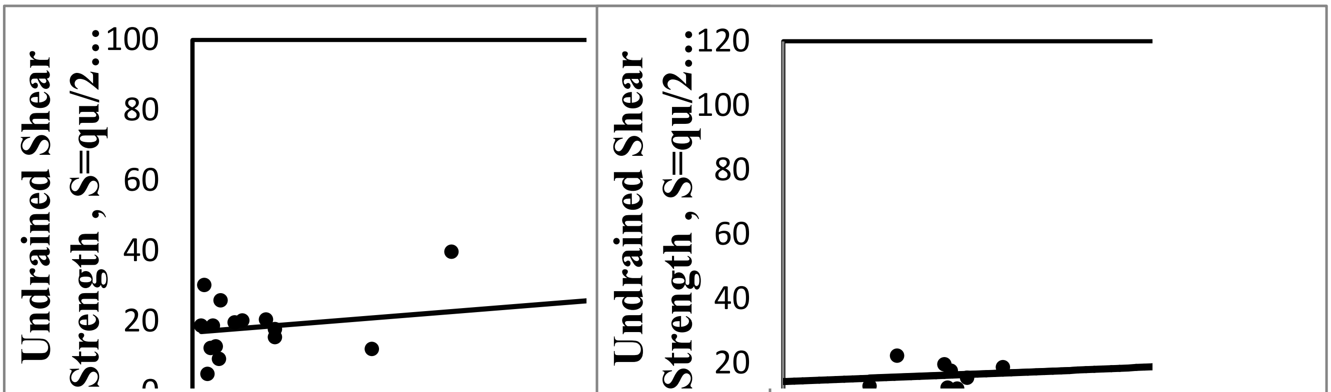


Figure-9: Effect of salinity and pH on Shear strength

From Figure-9, it is observed that the shear strength increases with increase of salinity and pH.

3.2.8 Variation of Initial Void Ratio with Salinity and pH

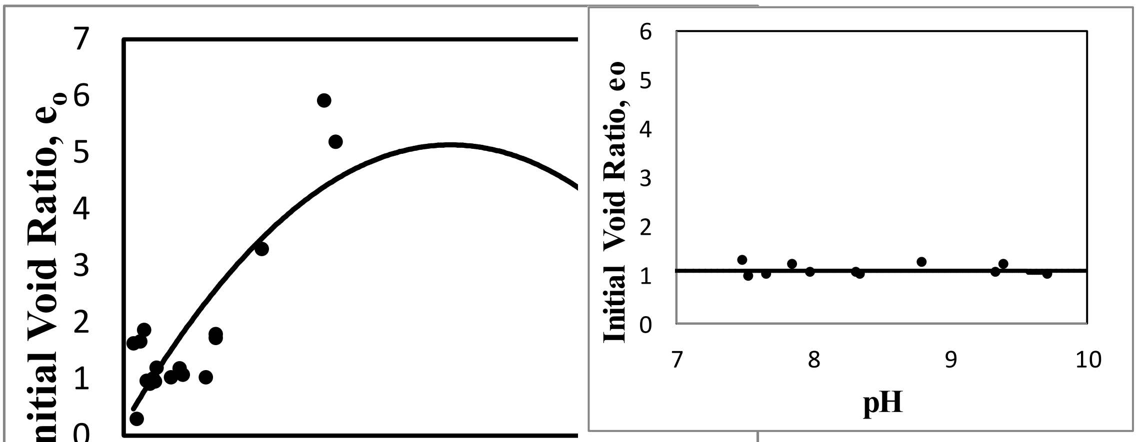


Figure-10: Effect of Salinity and pH on Initial Void Ratio

From Figure-10, it can be claimed that the variation of initial void ratio with pH is a convex shaped line. The initial void ratio increases with a peak when salinity is near about 5gm/L and after the peak it decreases. But the pH has no significant effect on initial void ratio of soil.

3.2.9 Variation of Compression Index with Salinity and pH

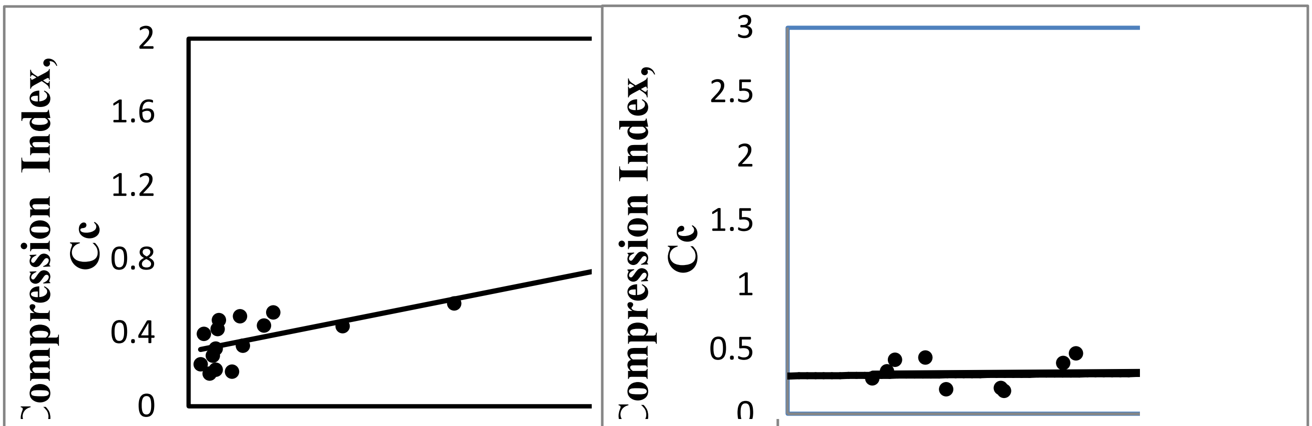


Figure-11: Effect of Salinity and pH on Compression Index

Figure-11 shows that the compression index increases with increase of salinity. But this property is almost independent of pH.

3.2.10 Variation of Recompression Index with salinity and pH

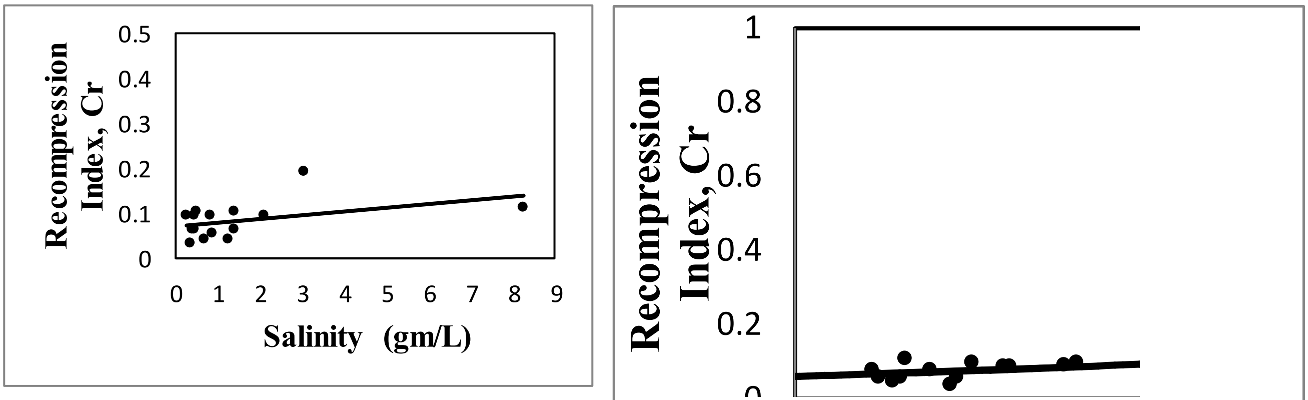
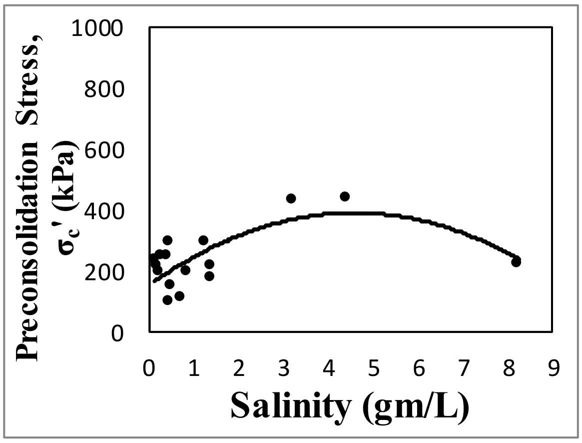


Figure-12: Effect of Salinity and pH on Recompression Index

From Figure-12, it can be said that the value of recompression index increases with increase of salinity .But the recompression index slightly increases with increase of pH.

3.2.11 Variation of Pre-consolidation stress with Salinity and pH



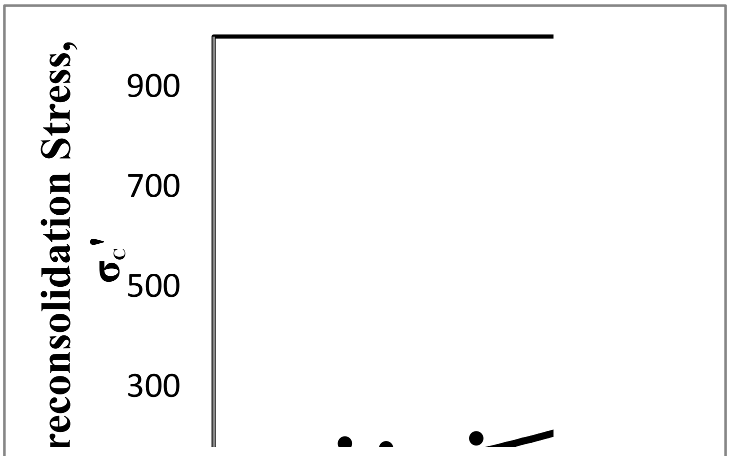


Figure-13: Effect of Salinity and pH on Pre-consolidation stress

From Figure-13 it can be said that the value of pre-consolidation stress increases with salinity up to the peak point near about 4.5gm/L and then it starts to decrease. Also the pre-consolidation stress increases with increase of pH.

4. CONCLUSIONS

The soil in the south-west region of Bangladesh has much salinity and pH. The geotechnical properties of soils of this region are changing from the past due to high salinity and pH content in the soils of this region. The salinity directly affects the consistency as well as strength properties of the soil. From the previous study it has been found that the physical and engineering property of soil changes based on the type of salt present. The variation is different for the different type of salt as well as the soil type. In the soil the salinity is mostly sodium chloride (about 85%), with lesser amounts of sulphate, magnesium, calcium and potassium in decreasing concentrations. In this study, only chloride ion content has been tested. From the study theoretical relationship of geotechnical properties with salinity and pH have been obtained. The effect of salinity and pH on geotechnical properties of soils has been enlisted in the Table-2.

Table -2: The variation of soil property with Salinity and pH

Soil Property	Increase / Decrease with Salinity	Increase / Decrease with pH
Specific gravity	Small variation (negligible)	Small / No variation
Moisture content	Decrease	Increase up to peak then decrease
Plastic limit	Decrease	Decrease
Shrinkage limit	Decrease	Decrease
Liquid limit	Increase	Decrease
Unit weight	Increase	Decrease
Undrained Shear strength	Increase	Increase
Initial void ratio	Increase up to peak then decrease	Small / No variation
Compression index	Increase	Small / No variation
Recompression index	Increase	Slightly increasing
Pre-consolidation Stress	Increase up to peak then decrease	Increase

REFERENCES

Chittaranjan, M., Srikanth, T., Lakshmi,B. Y.& D. Pavani. (2011). “Effect of sea water on some geotechnical properties of clayey soil.” International Journal of Earth Sciences and Engineering, V 4, 161-164.

Naeini S .A. and. Jahanfar M. A. (2011). “Effect of Salt Solution and Plasticity Index on undrain Shear Strength of Clays.” World Academy of Science, Engineering and Technology, 982- 986.
Salman, F. A., Sabre, D. K. and Namir, K. S. (2011). “Compressibility characteristics of saline soils treated with cement”. International Journal of the Physical Sciences, V 6(33), 7614 – 7628.
Sharma, P., Vyas, S., Mahure, N. V., Kachhal, P. I., Sivakumar, N., Ratnam, M. (2011), “Effect of acid rain on geotechnical properties of soils of high and low plasticity.” International Journal of Earth Sciences and Engineering, V 4, 158-160.
Umesha, T.S., Dinesh, S. V., and Sivapullaiah, P. V. (2012), “Effects of acids on geotechnical properties of black cotton soil.” International Journal of Geology, V 6, 69-76.

EFFECT OF AGGREGATE SHAPE ON THE STRENGTH OF BITUMINOUS MIXES

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ABSTRACT

Bangladesh is a developing country and its road network demands an ever-increasing expansion. Roads in Bangladesh are usually constructed of flexible pavement. The type and quality of aggregates are of prime importance on the design strength of the pavement. A laboratory investigation was made to evaluate the effect of the shape of aggregates as manifested by flakiness and elongation indices on the design strength of the bituminous mixes for flexible pavement construction. Test results revealed that both stability and flow values increased with the decrement of flaky and elongated particles for the aggregate matrix. At a temperature of 300^0F and when no elongated or flaky particles were used in the mix, the stability value was 340 kN and the flow value was 4.8. When temperature was increased to 320^0F, stability also increased by 18.5% but the flow reduced to 4.4. Stability value reached as high as 598 kN when flakiness index was 34% with no elongated particles in the mix.

Keywords:*Flakiness index; Elongation index; Stability; Flow; Strength.*

1. INTRODUCTION

Economic development of a country is indicated by the development of the communication infrastructure of the country. The present demand of the quick mobility of human and materials depends on the increase of motor vehicles. In Bangladesh, about 36% of the current total road motor vehicles of all types wereadded just in the last four years at an accelerating annual rate (BRTA, 2013). Consequently, the continual increase of pavement construction and reconstruction is a general need. In Bangladesh, the traffic phenomenon has gained tremendous importance of late. From only 3,600 KM of paved road at the pre-independence 1971throughout Bangladesh, it is now about 72,000 KM(LGED, 2013). If only the capital city is considered, an area of about 1463 sq. KM is seen to be consisting of 1700KMof paved roads. It is estimated that the space occupied by paved roads in Dhaka is about 9% of the total area whereas it is ideal to have about 25% of the area covered by the total street network. As a result, traffic jams and accidents happen frequently on the city roads. They are supposedly associated with the meagre space availability for more than643,000 motorized vehicles in conjunction with over one millionrickshaws apart from other non-motorized vehicles of various types(Alam, 2011;BRTA, 2011). A survey conducted for the road accidents showed that more than25,000 killed and another 12,000 injured apart from huge property damage and social disturbances in the last decade(BRTA, 2008). One of the factors leading to the occurrences of accidents was poor pavement conditions. Maintenance of pavements devoured a huge amount from the national exchequer per year due to surface defects, cracks, deformation, and disintegration. In addition, Bangladesh is not gifted with all the materials required for its infrastructure development; sometimes materials need to be imported from abroad. Therefore, a compromising balance must be made between the selection of materials and their mix design to ensure maximum durability and minimum recurring expenditure, keeping avenues for stage construction as the situation demands. There are still over 218,000 KM of earthen road connected to the paved road network throughout Bangladesh, are expected to be paved in the near future in phase construction (LGED, 2013). Thus the tremendous necessity for pavement construction and renovation in the city in particular as well as throughout the country in general cannot be overemphasized. Construction of flexible pavements is preferred in Bangladesh because of their low initial cost and adaptability for stage construction. A flexible pavement is a layered structure that receives the axle loads directly and transmits the same to the underneath layers without being overstressed. The transfer of load is influenced by the aggregate interlock, particle friction and cohesion for stability (Ioannides and Korovesis, 1990). The aggregates not only support the main stresses occurring within the pavement but also resist wear due to abrasion by traffic and weathering effects as well. The strength and performance of the pavement layer is directly related to the pavement structure which has direct bearing with the inherent properties and qualities of the individual particles and on the means by which they are held together i.e. by interlocking, by cementitious binder, or by both. The coarse aggregates for asphalt concrete should be hard, durable and angular. They are generally crushed rock and gravel. The strength of an aggregate and its resistance to abrasive wear and to polishing are determined largely by the properties of the parent rock from which the aggregate is derived. On the other hand, the overall strength of the compacted bituminous concrete and aggregate-bitumen bond would largely influenced by the aggregate shape and surface texture. The ratio of the least or greatest dimension of an aggregate to its mean size introduces two indices, namely flakiness and elongation, which is an important aspect to determine the stability of a bituminous mix. In order to produce a better mix considering the materials availability and environmental effects, this paper studies the effect of elongation and flakiness indices of the coarse aggregates on the strength of bituminous mixes.

2. EXPERIMENTAL PROGRAM

2.1 Materials and mix design

Aggregates from natural stone may have various shapes after crushing in the stone crusher.The particle shape of aggregates is determined by the percentages of flaky and elongated particles that it contains. Flaky particles are defined as those particles whose least dimension is less that 0.6 of their mean size while elongated particles are those whose greatest dimension is more than 1.8 times their mean size. Thickness gauge and length gauge conforming BS812were used to determine the flakiness and elongation indices of coarse aggregates. According to the British Standard (BS), single sized road stones and chippings require that maximum permissible flakiness index for 40 and 50 mm single sized stone does not exceed 40 and maximum permissible flakiness index for 10, 14, 20 and 28 mm single sized stone does not exceed 35. The maximum size of coarse aggregate used was 20 mm. The basic properties of the coarse aggregate are shown in Table 1.

Table 4: Basic properties of coarse aggregate

Type of test	Values
Flakiness index	34
Elongation index	27
Aggregate crushing value	36%
Aggregate impact value	25%
10% fines value	250 kN
Los Angeles abrasion	16%

Penetration grade bitumen 80/100 is normally used in Bangladesh for pavement construction, which conforms to the requirements of AASHTO M20-70 penetration grade asphalt cement grade 60-70(AASHTO, 1970). The basic properties of the bitumen are shown in Table 2.

Table 5: Basic properties of bitumen

Type of test	Values
Penetration	80-100
Flash point	290^0C
Fire point	315^0C
Specific gravity	1.02
Solubility	99.50%
Loss on heating	0.57
Ductility	100 cm

An optimum bitumen content was determined from five trial batches of bituminous mixes having varying bitumen content between 5 and 7%. The optimum bitumen content was then used to prepare six batches of bituminous mixes with varying contents of flaky and elongated coarse aggregates. Three types flaky aggregate contentnamely 0%, 15% and34% and three types of elongated aggregate content of 0%, 17% and 27% used to modify and adjust the content ofaggregate shape in the mixture gradation.Filler materials of 3% were used in all mixes. The mixture combinations are shown in Table 3.

2.2 Sample preparation and testing procedure

Correct design and physical testing of bituminous mixtures is important to construct sound, durable and economic pavements and to ensure minimum maintenance during the design life. A good bituminous paving mix should exhibit stability, durability, workability and skid resistance besides economy. Temperature of the mix, mixing procedure, compaction and physical testing all contribute towards the design of a suitable mixture. The Marshall method of mix design was developed by Bruce Marshall in 1948 and subsequently improved by the US corps of engineers (The Asphalt Institute, 1988). The Marshal method uses standard cylindrical test specimens of diameter 102 mm and 64 mm high. The specimens were prepared following a specified procedure for heating, mixing and compaction of the asphalt-aggregate mixtures. The two principal features of the Marshal method of mix design were a density-voids analysis and a stability-flow test of the compacted test specimens. The latter was followed in this research. The stability of the test was determined from the maximum load resistance in Newtons (N) that the standard test specimen would develop at 60^0C when tested. The flow was the total movement or strain, in units of 0.25 mm occurring in the specimen between no load and maximum load during the stability test.

Table 6: Aggregate combinations for bituminous mixes

Batch	Flakiness Index (%)	Elongation Index (%)
M1	0	0
M2	0	27
M3	15	27
M4	34	0
M5	34	17
M6	34	27

The load was applied to the specimen at constant rate of deformation until failure occurred. The point of failure was defined by the maximum load reading obtained. The total force (in N) required to produce failure of the specimen at 60^0C was recorded as its Marshall stability value. While stability was in progress, the flow meter was held firmly in position over guide rod and removed as the load began to decrease; reading was taken and recorded. That reading was the flow value for the specimen expressed in units of 0.25 mm. Two properties were determined – the ultimate load before failure (i.e. Marshall stability) and the amount of deformation at the ultimate load (Marshall flow). The ratio of stability to flow was denoted as the Marshall quotient, which gave a measure of the materials' resistance to permanent deformation.

3. RESULTS AND DISCUSSIONS

3.1 Optimum bitumen content

Relationship of Marshall stability and flow with five asphalt content of 5.0, 5.5, 6.0, 6.5 and 7.0% are shown in Figure 1. Stability values increased with the increase of asphalt content up to 6.0% and decreased thereafter, while flow of the mixture increased continuously with the increase of asphalt content as expected. Figure 2 shows the relationship of unit weight and amount of void (%) with varying asphalt content. Both the void (%) in total mix and in mineral aggregate decreased with the increase of asphalt content, however, the unit weight of the mix reached its maximum for an asphalt content of 6.5%. Thus the optimum bitumen content was the average of 6.0 and 6.5 i.e. 6.25. The parameters at optimum bitumen content were Marshall stability = 407.5 kN, flow = 0.045 in, unit weight = 2.27 gm/cc, void in total mix = 3.47% and void in mineral aggregates = 21.0%. The objective when designing continuously graded mixes is to achieve a dense mix of high stability but with sufficient voids between aggregates particles ensuring that sufficient bitumen is added to achieve flexibility, durability and workability without sacrificing resistance to permanent deformation.

3.2 Influence of elongation index

Stability-flow relationship with varying elongation index is shown in Figure 3. Both stability and flow value decreased with the increase in elongation index. The particle was said to be elongated when its dimension is greater than 1.8 times the mean size. With higher elongation indexof the aggregates the total contact surface area between aggregate and binder got reduced and amount of total void in the mix would be increased, which adversely affected the stability of mix. When there were no elongated particles in mix, the Marshall stability was 589 kN for a fixed flakiness index of 34% at a temperature of 300^0F. With a 17% increase in the elongation index the stability got reduced by 12.5%. However, the rate of strength reduction significantly slowed down with further increase in the elongation index. Only a less than 1% more reduction in stability was observed with further increase of elongation index by 10% i.e. at total of 27%.

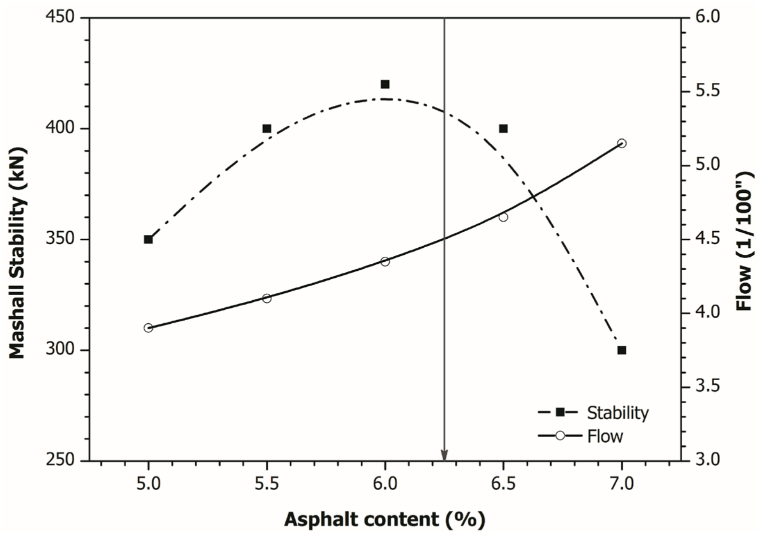

Figure 15: Optimum bitumen content from Stability-flow relationship

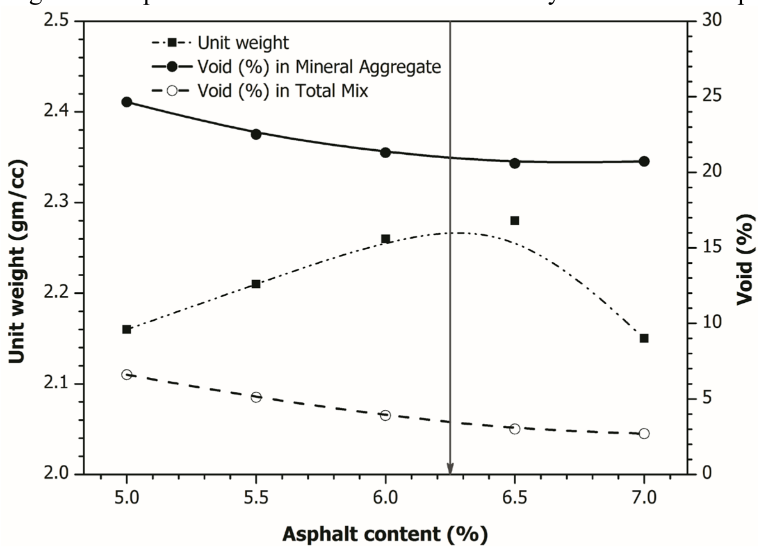

Figure 16: Optimum bitumen content from unit weight and void ratio (%)

Under traffic loading the layers of a flexible pavement structure are subjected to continuous flexing. The magnitude of strain was likely to depend on the overall stiffness and nature of the pavement construction. There are two categories of stiffness namely elastic stiffness under conditions of low temperature or short times of loading and viscous stiffness at high temperatures or long times of loadings. The former is used to calculate stains in the structure and the latter is used to assess the resistance of the materials to deformation. To determine the permanent deformation resistance of a bituminous mix, its response at high temperatures or long loading time was analyzed. Marshall stability at higher mixing temperature (320^0F) was decreased linearly with the increase in elongation index. They were about 3% lower than that at temperatures of 320^0F. It was reported that when the stiffness of bitumen was less than $5x10^6$ Pa, the mix behavior was much more complex than that in the elastic zone(Ehrola and Turunen, 1995).Figure 3 shows that the strain values (or Marshall flow in other words) showed negligible changes with the increase in elongation index up to 17%, however, decreased about 18% with further increase of the elongated particles.

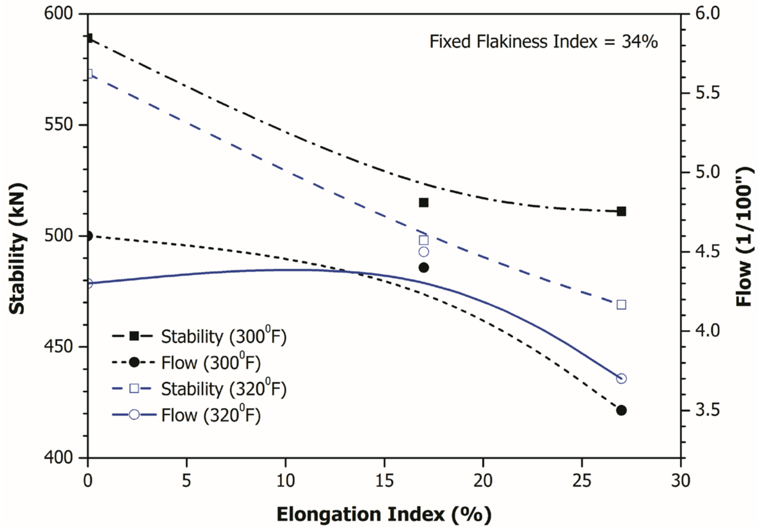

Figure 17: Marshall stability and flow for varying elongation index (%)

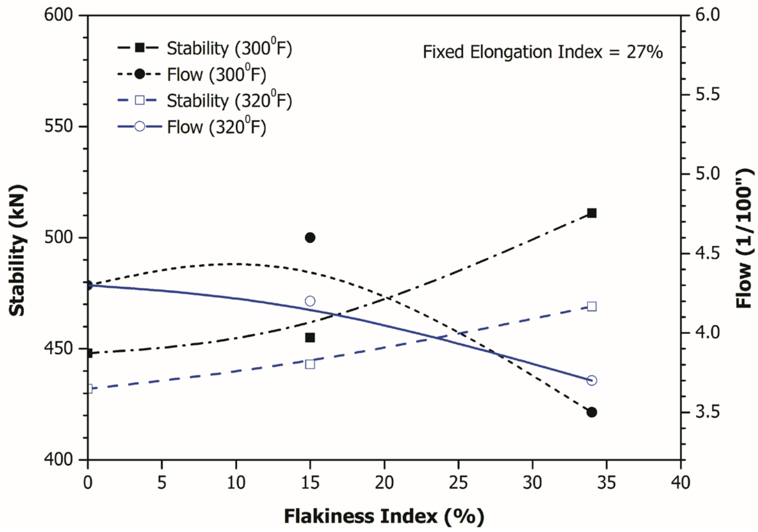

Figure 18: Marshall stability and flow for varying flakiness index (%)

3.3 Influence of flakiness index

The variation of Marshall stability and flow for varying flakiness index with a fixed elongation index of 27% is shown in Figure 4. With the increase in flakiness index Marshall stability of the compact increased steadily. The particle was said to be flaky when its least dimension or thickness was less than 0.6 times the mean thickness. When there was no flaky or elongated particles in the mix the stability values were 438 kN at temperatures of 300^0F. When the flakiness index was increased the total contact surface area also increased and amount of total void in the mix would be reduced. A combination of 34% flakiness index and 27% elongation index, as was the natural properties of the coarse aggregates under study (Table 1), yielded a stability value of 511 kN at 300^0F. Although it was about 13% less than that produced by the mix with no elongated particles (Figure 3), mixes with abnormally high values of Marshall stability and abnormally low flow values are often less desirable because pavements of such mixes tend to be more rigid or brittle and may crack under heavy volumes of traffic. This is particularly true where base and subgrade deflections are such as to permit moderate to relatively high deflections of the pavement. Stability at 320^0F increased linearly with the increase in flakiness index and was consistently 3% lower than that for 300^0F up to a flakiness index of 15%. It reached 469 kN at a flakiness index of 34%.

Flow value of the compact increased slightly with increase in the flakiness index of 15% but started to decrease afterwards. As the flow value increased the mix became unstable, but when the flakiness index increased further, the flow value decreased, which was an indication of a stable mix. However, when more flaky particles were present in the mix, the specimen was initially vertically deformed to a certain amount during the blowing (compaction) period. Thus it is obvious that a certain amount of flaky particles were needed to have a stable mix.

4. CONCLUSION

This paper studied the effect of flaky and elongated coarse aggregates on the strength of bituminous mixes for flexible pavement in terms of Marshall stability and flow value. The bitumen content was kept same with 3% filler in all mixes, while temperature was varied between 300 and 320^0F. The following conclusions are drawn:

- Highest stability of 589 kN was obtained from a combination of 0% elongation index with 34% flakiness index, while lowest flow of 0.035 in. was obtained from a combination of 27% elongation index with 34% flakiness index.
- Increasing elongation indices reduced the stability value up to about 13% for a fixed flakiness index.
- Marshall stability was found to increase up to about 14% with the increase in flakiness indices by 34% for a fixed elongation index.
- Elongated particles caused more instability than that by the flaky particles in terms of Marshall flow.
- A flakiness index of higher than 15% deemed necessary to have a stable bituminous mix.
- At higher mixing temperature (320^0F) didn't affect the flow value but reduced the stability 3-8%.

REFERENCES

AASHTO (1970). "Standard Specification for Penetration-Graded Asphalt Cement", AASHTO M20-70, American Association of State Highway and Transportation Officials, Washington, DC.

Alam, H., "Rickshaws clog city", *in*: The Daily Star, Dhaka, Bangladesh, March 29, 2011, http://archive.thedailystar.net/newDesign/news-details.php?nid=179563.

BRTA (2008). "National Road Traffic Accident Report", Bangladesh Road Transport Authority, Dhaka, Bangladesh, http://www.brta.gov.bd/index.php/road-safety.

BRTA (2011). "Registered Vehicles in Dhaka", Bangladesh Road Transport Authority, Dhaka, Bangladesh, http://www.brta.gov.bd/media/com_acymailing/upload/motor_vehicles_dk_06_09_11.pdf.

BRTA (2013). "Registered Vehicles in Bangladesh: 2009 to 2013", Bangladesh Road Transport Authority, Dhaka, Bangladesh.

Ehrola, E. and Turunen, R. (1995). Bituminous pavements: materials design and evaluation. University of Oulu, Oulu, Finland.

Ioannides, A.M. and Korovesis, G.T. (1990) "Aggregate interlock: a pure-shear load transfer mechanism", *in*: Design and evaluation of rigid and flexible pavements, Transportation Research Record, Transportation Research Board, Washington, DC 20001 USA, pp. 14-24.

LGED (2013). "Road Database", Local Government Engineering Department, Dhaka, Bangladesh, http://www.lged.gov.bd/ViewRoad2.aspx.

The Asphalt Institute (1988). "Mix design methods for asphalt concrete and other hot-mix types", Manual Series No. 2 (MS-2), The Asphalt Institute.

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COST OF IRRIGATION WATER IN BANGLADESH: A STUDY FROM CHITTAGONG AND RANGPUR DISTRICT

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ABSTRACT

Both ground and surface water is becoming scarce with time due to ever increasing demand from agricultural, industrial, municipal and environmental sectors, which results an increase in cost to water supply. The scenario is common all over the world and particularly in Bangladesh. This paper aims to estimate the cost of irrigation water in Bangladesh particularly for the Boro rice, which contributes major share in rice production consuming the largest amount of irrigation water. Both ground and surface water is used for irrigation to Boro field based on the availability of the water. The study was carried out in two villages from Chittagong and Rangpur district, one in each, representing both types of irrigation sources. Cost of irrigation water is estimated based on data obtained through primary survey administered to the farmers in the study areas in June and July 2013. Study site in Chittagong experiences surface water irrigation mainly coming from the Karnafuly whereas farmers at the Rangpur site use groundwater. Analyses show that cost of surface water irrigation is about Taka 1.21 per m^3 but cost of groundwater irrigation is about Taka 1.92 per m^3. Even though the yield in Rangpur site is higher than Chittagong site, due to higher cost of irrigation water, famers in Rangpur make a lesser profit of about Taka 2,044 in compare to the farmers at Chittagong site. Such analysis and results will be helpful for formulating price for irrigation water supplied by Bangladesh Water Development Board as well as other private irrigation water suppliers. In addition the study will be helpful for irrigation management and ultimately it will contribute to ensure food security of the country.

Key words: cost of surface water irrigation, cost of groundwater irrigation, Bangladesh.

INTRODUCTION

For food security, irrigated agriculture has been an essential factor for last few decades. But in Bangladesh, for the rapidly growing economic activity there are increasing concerns of water scarcity. About eighty six percent of all water use occurs in the country only for agricultural purpose, major of it in the form of groundwater irrigation and minor of them in the form of surface water irrigation. Population is increasing day by day and subsequently food demand is increasing. As a result, it has become very important to identify the irrigation water demand and its cost in Bangladesh so that irrigation management and food production can be more effective. This paper aims to determine the cost of irrigation water based on data obtained from a primary survey administered to the study areas representing one groundwater irrigation site and another surface water irrigation site. With the collected data from the survey a comparative analysis was done of cost of using surface water and groundwater for irrigation purpose.

STUDY SITES

In this research, the area “Chora-bot-tol” of chittagong district is considered as “Study area-1”. The area is located in Chittagong district in the Ranguniya upazilla and in the union of Chondroghona. In that area the survey was done for surface water irrigation cost. The area was selected for the analysis of surafce water irrigation system because the irrigation of the area is mainly based on the water of the river Karnaphully. The irrigation is done with pumpnig water from the river. There are two types of machines used for this purpose. One kind is operated with diesel and the other kind is operated with electricity. Now a days only diesel machines are used. Elecrical machines were used around 15 years ago.

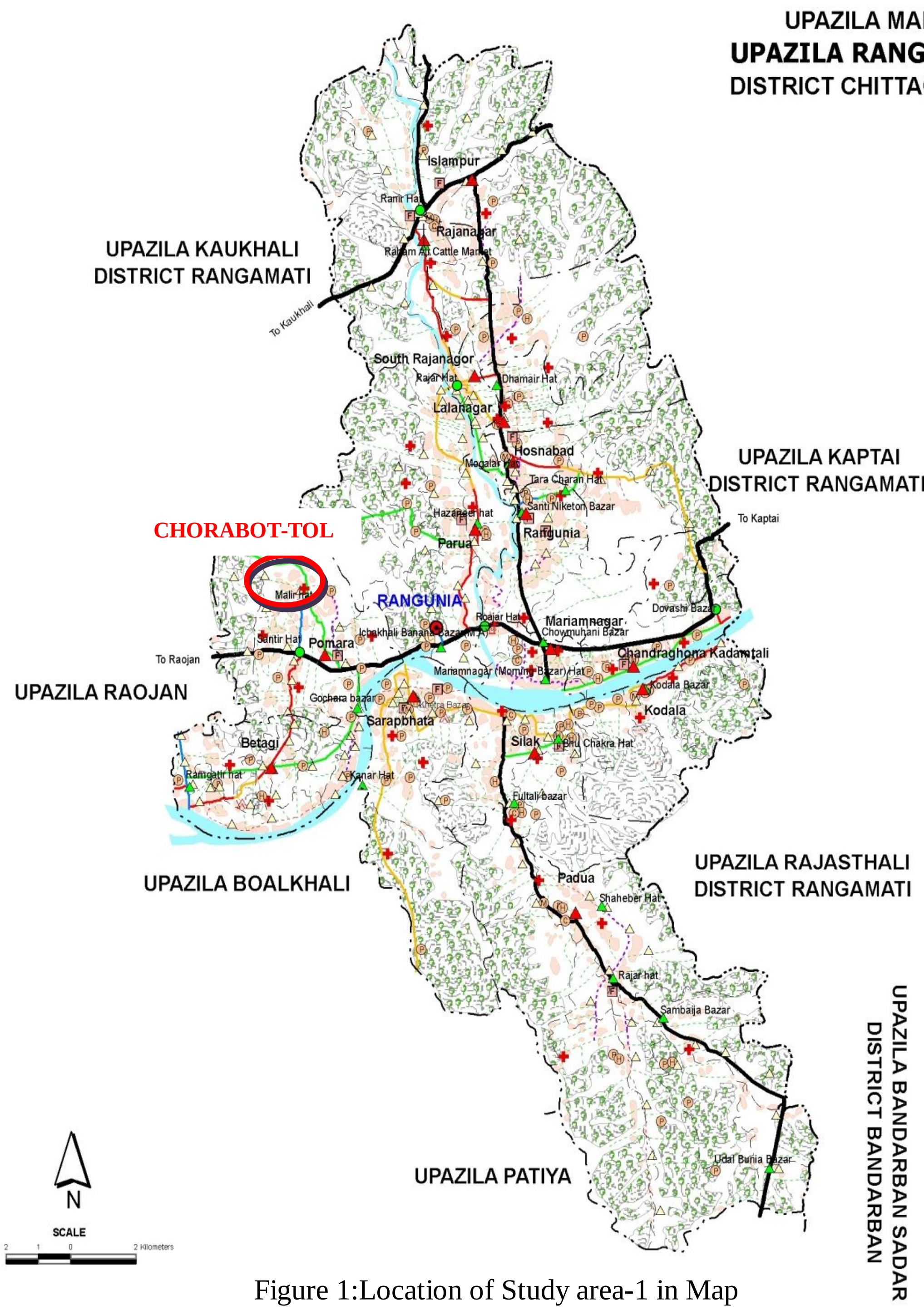

Figure 1:Location of Study area-1 in Map

On the other hand, the area of Parghat under the union Mominpur, ward no:4, of the district Rangpur was selected as 'study area-2''. The population of the area is 1500. There is high class (10%), middle class (60%), low class people (30%) in the area. The land of that area is moderately plain. The area is not well developed. The main crop of that village is rice. But some other crops are also grown, namely corn, sugarcane, jute, potato and tobacco.

In this area, the analysis was done for the investigation of cost of ground water irrigation. The irrigation of that area is mainly based on deep tubewell pumping. Though there is a river named "kharvuj" in the village, but it is not used for irrigation purpose as the water level of the river is very low and there is not sufficent water available.

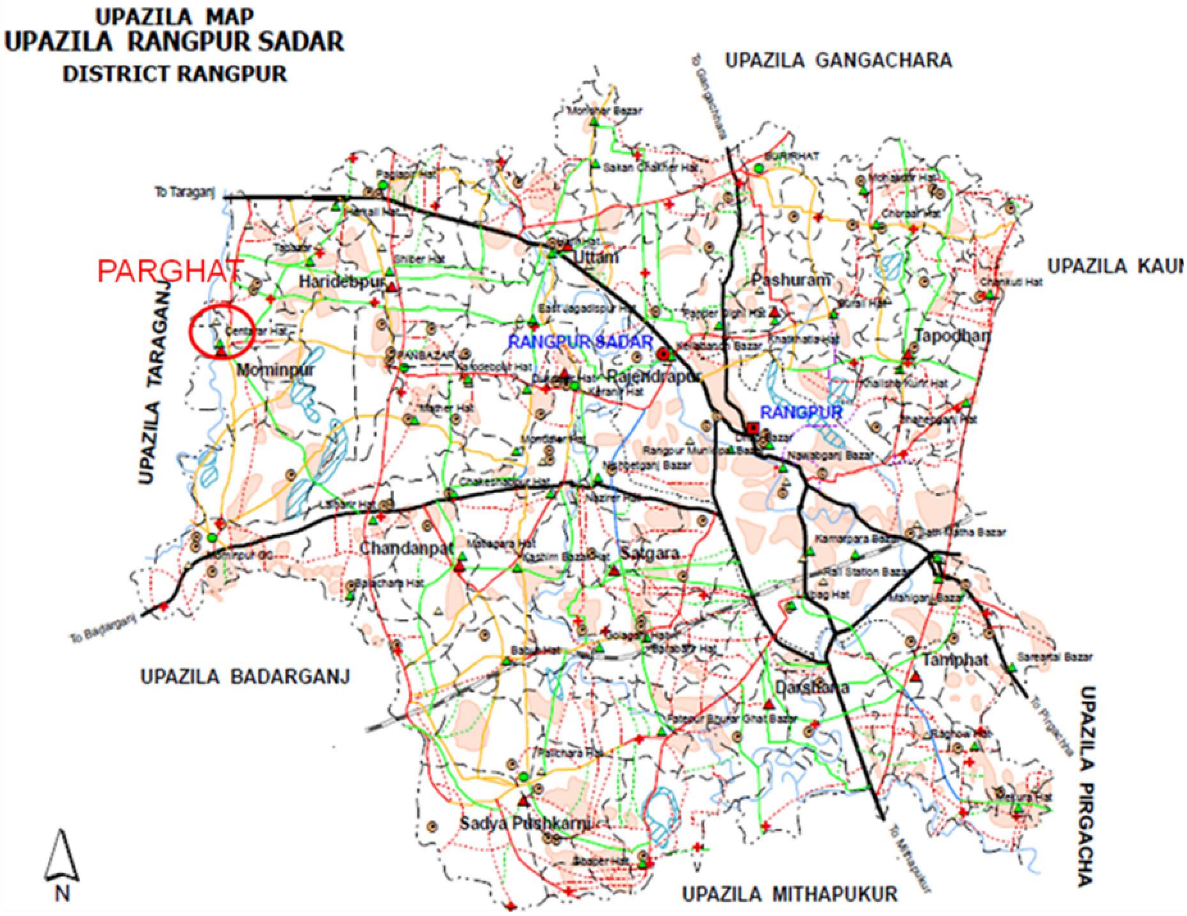

Figure 2: Location of study area-2 in Map

METHODOLOGY

There are mainly two seasons of rice cultivation all over Bangladesh. The first season starts from mid December to mid May and the second season starts from mid August and continues till November. The “Boro” rice in cultivated during the first season and in the second season “Aman” rice is cultivated. As there is sufficient rainfall during the second season, minimum additional water is required for the cultivaton of “Aman” rice. The analysis therefore was done only for “Boro” rice.

COLLECTION OF DATA

In the analysis of cost of bith ground and surfacewater irrigation, the water requirement of rice crop in different stages was first investigated.The duration of stages that has been considered for both analysis is summarized below in Table 1.

Table 1: Duration of different stages of rice cultivation

Stage	Duration
Nursery	1 to 2 months (usually 1 month)
Land preperation	3 days(included in the duration of total growth)
Growth of plants	90 to 120 days
Harvestin g	Depends on many factors

RESULT

Surface Water Irrigation System at Chittagong

Collected data on watering interval and payment by the farmers are presented in Table 2.

Table 2: Interval of watering and payment system in different stages of rice cultivation for surface water irrigation

Stages	Watering interval	Payment per season per hectare
Nursery	2 times	3088 taka
Land preperation	1 time only	No extra payment is required
Growth of plants	5 days	9264.9 taka
Harvesting	No water is required	-

*Taka 76=1USD

Estimation of Cost of Irrigation

1 hectare= 10000 m^2

The Net irrigation requirement is 1019 mm = 1.019 m as estimated by Mullick et al. (2011)

The volume of water required per hectare = 10190m^3

To buy 10190 m^3 water, total 12,352.9 taka per season is required.

To buy 1 m^3 water 1.212 taka per season is required

Benefit Calculation

From the field investigation, it was observed that, In one hectare land the farmer has to invest taka 55,589 per season including water, fertilizer, pesticides, labour, seeds per season In 1 hectare land, about 4941 kg rice is grown.

The market price of this kind of rice is taka 17.5 per kg

The net profit from 1 hectare land is 30,878 taka

Ground Water Irrigation System at Rangpur

Collected data on watering interval and payment by the farmers are presented in Table 3.

Table 3: Interval of watering in different stages of rice cultivation for ground water irrigation

Stages	Watering interval	Payment per season per hectare
Nursery	3 times	4119 taka
Land preperation	1 time only	No extra payment is required
Growth of plants	3 days	15,447 taka
Harvesting	No water is required	-

Estimation of Cost of Irrigation

To buy 10190 m^3 water, total 19566 taka per season is required.

To buy 1 m^3 water 1.92 taka per season is required

Benefit Calculation

From the field investigation, it was observed that, 1 hectare land the farmer has to invest taka 51,493 per season including water, fertilizer, pesticides, labour, seeds per season.

In 1 hectare land, about 6179 kg rice is grown

The market price of this kind of rice is taka 13 per kg

The net profit from 1 hectare land is 28,834 taka.

DISCUSSION

From the above analysis, it is seen that there is significant difference in the cost of ground water irrigation and cost of surface water irrigation. The net irrigation requirement is again dependent on humidity, temperature, rainfall and moisture content of the soil concern; however, in this analysis net irrigation requirement for both the study sites are considered equal. In our analysis, the cost of ground water is observed higher than the cost of surface wate for unit m^3. Nevertheless, the benefit made by using surface water irrigation is higher than the benefit made by ground water irrigation per hectare. The results are summarized and compared in Table 4.

Table 4: Comparison of surface water irrigation situation and ground water situation

Subject of comparison	Surface water irrigation	Ground water irrigation
Total Taka needed to buy $1m^3$ water during the crop period	1.2 taka per season	1.9 taka per season
Net benefit from 1 hectare land	30,878 taka	28,834 taka

The principal crop of Bangladesh is rice and that is a water intensive crop. This research attempts to understand the comperative price of the two irrigation system, based on the irrigation of that crop**.** Farmers in Bangladesh do not pay for the use of per unit volume of irrigation water. When surface water is abundant farmers solely depended on rivers, canals and ponds to irrigate their fields with traditional local methods where the maintenance cost of the apparatus and labour charges were the costs of irrigation. For the government run canal irrigation methods the farmers usually pay a fixed amount per unit of irrigated land per crop during the season and in case of participatory water management water user groups pay for maintenance of field channels plus management cost in some cases. It varies from case to case. With the advent of groundwater irrigation cost of irrigation consists of maintenance of the pumps, fuel cost (electricity or diesel) and the salary of the pump mechanic when required. Farmers use low lift pump to pump water from surface water sources and shallow or deep tube wells for groundwater. The use of low lift pump is limited by the availability of surface water in the canals and rivers during the dry season. Since investment in deep tube well is lumpy in nature farmers prefer shallow tube wells. In case of shallow tube wells the energy cost (electricity or diesel) is the main component of irrigation cost. As electricity is not available in all the villages farmers have to depend on diesel to run irrigation pumps to a large extent. Hence the price of diesel in the international market plays a crucial role in cost of irrigation for the private sector

RECOMMENDATIONS

The main challenge of public sector irrigation institutions is to design proper incentives for all stakeholders to participate in the participatory water management network. Successful water management practice for irrigation will depend on equitable participation of all groups of farmers as water user groups in management and cost recovery. Some necessary steps that we can recommend from our thesis are listed below:

1. Introduction of rice varieties that require less water for irrigation per hectare is essential to reduce water cost. Government should give incentives or price support for wheat and maize production so that farmers diversify towards these crops that require much less water per hectare for irrigation compared to “Boro” rice.

2. In order to run the pumps with electricity, stability in power supply is a must that will reduce the cost of irrigation as well as cost of cultivation drastically. In this endeavour there is no alternative to 100 percent rural electrification.

3. Bangladesh Water Development Board should be aware of the cost of irrigation and they can fix the maximum price of selling water, so that the water owner of different areas can not sell water in high rate.

REFERENCES

Mullick, M. R. A., Babel, M.S. & Perrer S.R. 2011. Discharge-based economic valuation of irrigation water: evidence from the Teesta River, Bangladesh. *Irrigation and Discharge*, 60(4): 481-492.

PROCEEDINGS

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