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HYDRAULIC ANALYSIS OF FLOW OVER A COASTAL EMBANKMENT

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ABSTRACT

Bangladesh delta, located alonside of the three trans-himalayan rivers- the Ganges, the Brahmaputra, and the Meghna, has a long history of flooding of low-lying coastal belt. This river delta is one of the largest in the world. Coastal embankments, named as Polder, were constructed during 60's to protect low-lying coastal areas from flooding. Over the years, poor maintenance and cyclonic storm surges have caused damages some of these polders, threatening about 42 million coastal residents and their communities. Moreover, these polders have been more vulnerable due to recurrent adverse effect of climate change like storm surges and sea level rise. Hydrodynamics of flow over an embankment is one of the severe hydraulic actions causing erosive damage along its countryside slope. Therefore, hydraulic analysis of flow over such embankment structure needs proper attention so that it becomes more safe and resilient to hydraulic action. This paper deals with a simple and systematic approach for estimating these hydrodynamic parameters of flow over typical coastal polders. Flow rate, overtopping flow depth, sequent depths of hydraulic jumps, shear stresses for various water level conditions have been estimated using the available approach and formulae. Necessary water level and typical polder dimension has been collected from BWDB and other sources. The hydraulic parameters obtained from this procedure can be useful to the designer dealing with improvement of coastal embankment.

Key words: *Flood overflow, Coastal embankment, Hydraulic data, Sequent depths, Shear stresses.*

1. INTRODUCTION

Once floodwater overtops an embankment, erosion of the embankment will occur when locally high velocities over the embankment create a high erosion force which exceeds the strength of embankment resisting erosion. The primary mode of embankment failure due to flood overtopping begins by erosion of the downstream shoulder and slope. The design and construction of overtopping protection for dams is increasingly being viewed as a viable alternative to constructing larger polder height by raising the crest. The decision to pursue overtopping protection for a dam must give strong consideration to the risk of failure of the protection system, which could lead to a full breach of the dam. Overtopping protection should generally be reserved for situations with a very low annual probability of operation, and with physical or environmental constraints and a prohibitive cost of other flood protection alternatives. Hydraulic, structural, and geotechnical engineers, as well as geologists, should be involved throughout the design process.

A polder is a low-lying tract of land enclosed by embankments (barriers) known as dikes that forms an artificial hydrological entity, meaning it has no connection with outside water other than through manually operated devices. Polder 32 (Figure 1b) is one of hundreds of low-lying islands located along the coastal plain of the Ganges-Brahmaputra delta. The area is whipped by cyclones in the spring and fall that drive surges of saltwater up from the Bay of Bengal. Summer brings monsoons that provide nearly 80 percent of the area's annual rainfall and can cause extensive flooding. There are three types of polder: i)Land reclaimed from a body of water, such as a lake or the seabed ii) Floodplains separated from the sea or river by a dike iii)Marshes separated from the surrounding water by a dike . Polders are at risk from flooding at all times, and care must be taken to protect the surrounding dikes. Dikes are typically built with locally available materials, and each material has its own risks: sand is prone to collapse owing to saturation by water; dry peat is lighter than water and potentially unable to retain water in very dry seasons.

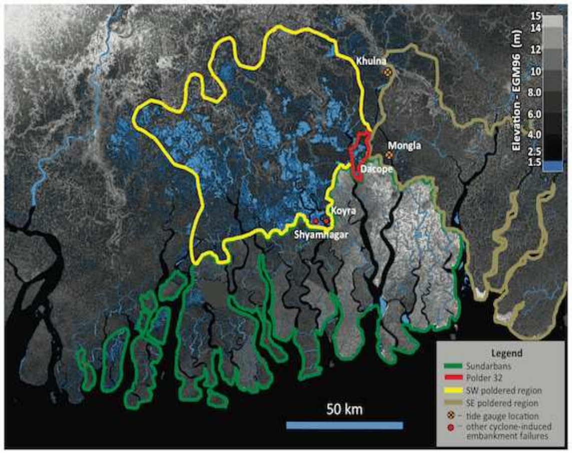

Figure-1(a) : Coastal polders and Polder No. 32

Figure-1(b): Flow overtopping polders (PPTA,2013)

Polders are most commonly, though not exclusively, found in river deltas, and coastal areas. Polder 32 is one of hundreds of low-lying islands located along the coastal plain of the Ganges-Brahmaputra delta. The area is whipped by cyclones in the spring and fall that drive surges of saltwater up from the Bay of Bengal. Summer brings monsoons that provide nearly 80 percent of the area's annual rainfall and can cause extensive flooding. Government of Bangladesh is paying for the project—known as Coastal Embankment Improvement Project, Phase-1 (CEIP-1) funded by World Bank. It is now clear to all that climate change has greater effect on coastal flooding (Ali, 1996). Under scenario A2 in year 2050 (SLR 27 cm) exposed population will be 5.0 million to high risk (inundation over 100 cm) and while 5.5

Figure 2(a): Flood over flow and hydraulic jump in nature

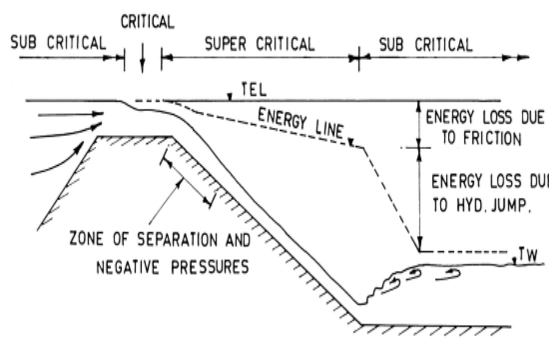

Figure 2(b): Definition sketch of flow over embankment

million at risk of inundation by 50-100cm. In case of 13 polders overtopped due to 62 cm sea level rise in the year 2080 under A2 scenario and 45% population will be exposed to medium to severe inundation (IWM & CEGIS, 2007).

2. APPROACH AND METHODOLOGY

2.1 Discharge Equation

The generally accepted form of the equation that computes discharge over an embankment for the free flow condition is

$$q = C\ L\ H^{3/2} \qquad (1)$$

Where q is the discharge per unit width, C is a coefficient that has been determined experimentally by a number of laboratory tests and modeling study. The value of C varies between 2.10-2.25 (Margeirsson, 2007). H is the total head above the embankment crest. where L represents the length of inundated embankment or roadway. When the depth of flow varies along the embankment structure, it is advisable to divide the inundated portion into reaches and compute the discharge over each reach separately.

2.2 Water Surface Profile

Over-flow depth at the brink of the crest and along the sloping side have to be estimated using the principles of gradually varied flow (GVF) and steady uniform flow respectively (Chow, 1959). Since the length of GVF profile S2 is not long enough and streatched near the crest, only flow depths along the sloping portion have been considered. It is assumed that the flow along the sloping part is uniform.

2.3 Sequent Depth Ratio of Hydraulic Jump at Toe

Applying both momentum, continuity equation, and introducing non dimensional terms for sequent depth ratio $D = y_2/y_1$ and for inflow Froude number $F_1 = U_1/(gy_1 \cos\Theta)^{1/2}$, in which $U_1 = q/y_1$, Θ = country side slope angle, a expression can be derived (Matin et al. 2008). The sequent depth ratio D is obtained as follows:

$$D = 0.5\left[(1+8G_1^2)^{1/2} - 1\right] \tag{2}$$

Where, G_1 = modified Froude number. The relationship between G_1 and F_1 can be rearranged as

$$G_1^2 = \frac{F_1^2}{k_1\left\{(1-k_2)\frac{\cos\theta}{k_1} - k_3\right\}} \tag{3}$$

Experiments had been conducted (Matin and Sultana, 2015) to obtain working relation of k_1, k_2 and k_3. If the sudden drop (Δz) at the toe of embankmement is disregarded, then $k_3=0$. The relationship representing k_1 and F_1 can be obtained as follows:

$$k_1 = -13.3 \ln(F_1) + 50.83 \tag{4}$$

Similar to Equation 4, the following regression equation for k_2 was obtained:

$$k_2 = 0.042F_1^2 - 0.275F_1 + 1.037\cos\Theta \tag{5}$$

2.4 Erosion Rate

When flow overtops an embankment, locally high velocities and shear stresses will create strong erosion forces, typically at the downstream shoulder and on the embankment slope, that are too great for the soil of the embankment to withstand. Hydraulic stresses less than the bed shear strength will cause minimal erosion. As shear stress on the bed increases beyond the bed shear strength, erosion from the surface of the bed will proceed at a rate that is proportional to the excess of the bed shear stress. Erosion rate is directly proportional to shear stress and that shear stress is proportional to the square of velocity. In the most simplified form the relationship between erosion rate and velocity is given by

$$€ = K(V^2 - V_c^2) \tag{6}$$

Where € is erosion rate kg/s/m, V is the over flow velocity m/s, V_c is the critical velocity (m/s), and K is the constant and conversion factor. For practical cases the following formulas can be used to estimate the critical velocity (V_c) causing soil erosion:

$$V_c = \sqrt{(g\Delta D_{50}\psi_{cr})}\, 5.75 \log\frac{6h}{D_{50}} \quad \text{(for finer soil)} \tag{7}$$

$$V_c = \sqrt{(g\Delta D_{50}\psi_{cr})}\, 5.75 \log\frac{2h}{D_{50}} \quad \text{(for coarser soil)} \tag{8}$$

Where, h is the depth of flow, Δ is the relative density of soil, ψ_{cr} is critical Shield's parameter. The critical mean velocity near the bed assuming uniform flow and smooth boundaries with uniform materials in the range of 0.1 mm < D_{50} < 1 mm can be estimated up to 0.2 m/s. Based on the worst case of sands subsoil the permissible mean velocity for general use can be taken as 0.5 m/s. Cohesive sediment such as clay usually has high resistance to erosion than non-cohesive sediment, V_c = 0.8 m/s – fairly compacted clay and V_c = 1.5 m/s - stiff clay (BWDB, 2008).

3. DATA

Data of the coastal water level and structural information have been collected from Coastal Embankment Improvement project-I (CEIP, BWDB). Information about sea level rise (SLR) has been obtained from the secondary sources. The embankment crest levels were checked against the 25 years return period maximum storm surge level and monsoon flood levels considering 50 cm sea level rise and increased precipitation projected by IPCC and the required free board for wave run-up considering 10% wind speed increased. 5 l/m/s overtopping and 30% land subsidence also considered for the crest level design (Rimon et al. 2014). The potential impacts of climate change and response strategies as per 4th IPCC (IPCC, 2007), the global prediction of 18 – 59 cm sea level rise from 1990 to 2100 59 cm. According to the Synthesis Report of Copenhagen Summit on March 2009 maximum SLR will be 1±0.5m by 2100 and thus SLR of 1 m have been selected for this Study.

The crest level and embankment cross sections have not optimized the protection of hinterland and the embankment itself and therefore the embankments typically only provide protection for the cyclones with 5-12 year return periods and the designed crest level of the sea facing coastal polder equal to the sum of normal maximum recorded water stage plus 1.50m (Rimon et al. 2013). Appropriate surge height to estimate the inundation zones through the equation outlined by Nicholls (2006). Estimated Future Surge Height,(FSH) can be obtained as follows:

$$FSH = SH + SLR + (UL * 100\ yr) / 1000 + \Delta z + RH\ x \tag{9}$$

Where, SH = Representative cyclone surge height (m); SLR = sea-level rise (m); UL = continental uplift/subsidence in mm/yr (Not accounted) ; Δz = 0.5 m (applies to deltas only), x = 0.1 or increase of 10%, applied only in coastal areas currently prone to cyclones. Current storm surge =2.75 m above the normal astronomical tide for cyclone Aila, 2009. Halder and Zaman (2011) made a prediction of storm surge height as shown in Figure 3.

Based on these information the water height ranges between 0.5m and 3.5 m above the embankment crest have been considered for possible hydraulic analysis. Thus the flow conditions overtopping the embankment include: i) Overtopping depths, H = 0.5, 1.0, 1.5, 2.0, 2.5, 3.0 and 3.5m. and ii) Tail water drop = free-fall conditions, considering the countryside (C/S) of the polder is flood free.

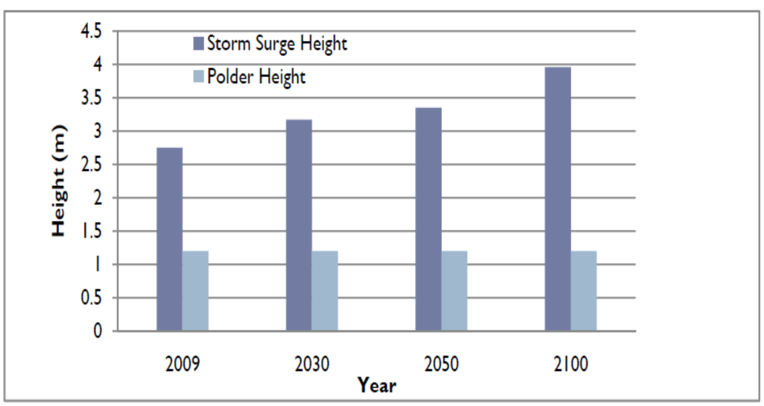

Figure 3: Predicted storm surge height and polder height

Table 1 summarizes the data used for necessary hydraulic calculation. Analyses have been done for per unit length of the embankment.

Table 1: Input data for hydraulic analysis

Input Data	Value
Over flow water depth range (m)	0.5 - 3.5
Length of the embankment	Unit length
Countryside side slope	1V:2H
Countryside soil type	Silty clay
Relative density	1.65
Soil size (mm)	0.10
Shield's parameter	0.03
Coefficient K for erosion rate	0.01

4. ANALYSIS, RESULTS AND DISCUSSION

4.1 Discharge Intensity

As stated in Art.2.1, the flow rates for various head over the embankment crest have been estimated using the well known weir formula. Rating curve (q vs. H) is shown in Figure 4. It shows that estimated discharge range 0.50 m^3/s and 14.5 m^3/s per meter length of the embankment corresponding head (H) of 0.5m and 3.5 m. These data have been utilized for various hydraulic calculations as discussed below.

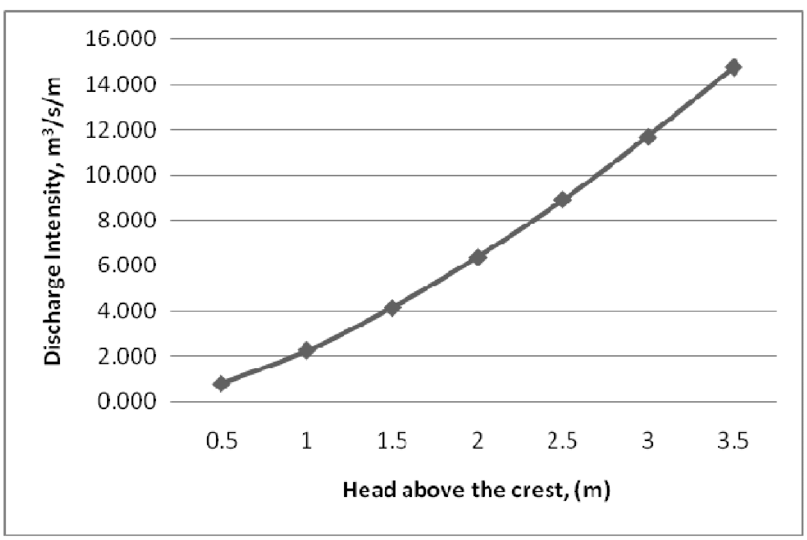

Figure 4: Discharge intensity vs. head above the crest

4.2 Flow Characteristics and Sequent Depths

Flow type over the embankment crest has to be known. Initially the flow over the crest crossed the critical depth and the flow should obviously be strongly supercritical nature with Fr_1=2.8 to 4.00 as demonstrated in Figure 5. When the flow overruns the sloping side of the embankment it becomes even more stronger supercritical with Fr_1 as high as 4 to 6 depending on the amount of flow that overruns the embankment. The approaching flow along the slope of the embankment hits at the toe where grade becomes horizontal. A strong hydraulic jump occurs and corresponding sequent depths have been computed by using the procedure as described in Art.2.3. Figure 6 shows the computed various depth versus flow head over the crest. It shows that the sequent depth (tail water) could be as high as 7.8 to 5.3 times the approaching depth. Such high energy flow could cause sudden disaster if proper remedial measures are not suggested in embankment design in coastal zone.

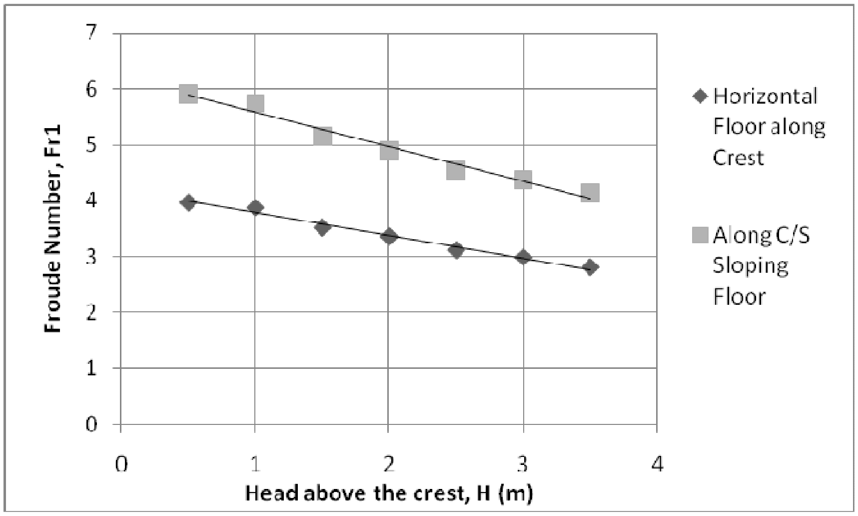

Figure 5 : Froude No. vs head over crest

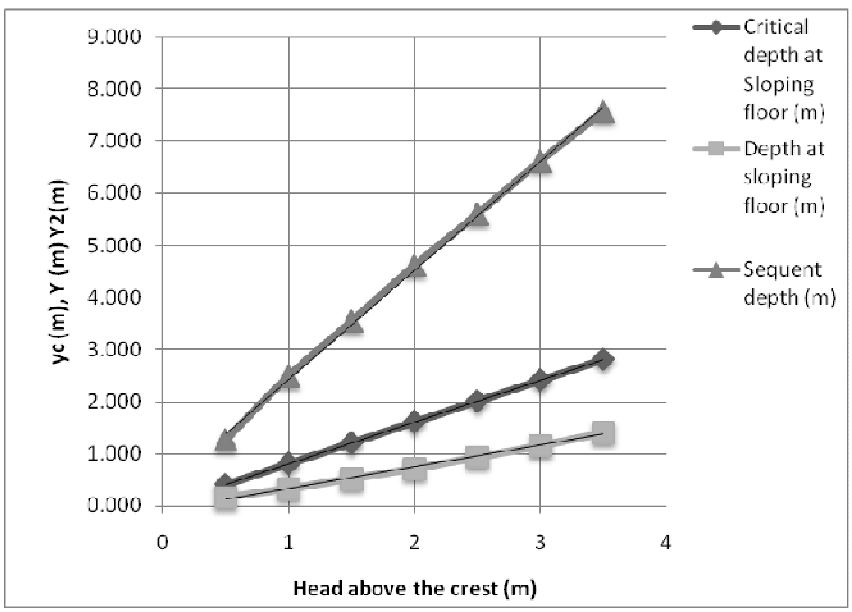

Figure 6 : Various depths over the sloping side vs. head over crest

4.3 Shear Stress and Impact Stress

As the flood flow over runs the countryside sloping portion of the coastal embankment, there exist tremendous shear stress along the slope. Calculation shows that for given range (0.5 m - 3.5 m) flood flow head, the shear stress varies between 0.78 KN/m to 6.9 KN/m per meter length and corresponding impact force of water flow at the vicinity of the toe range between 3.95 KN to 159.94 KN (Figure 7)

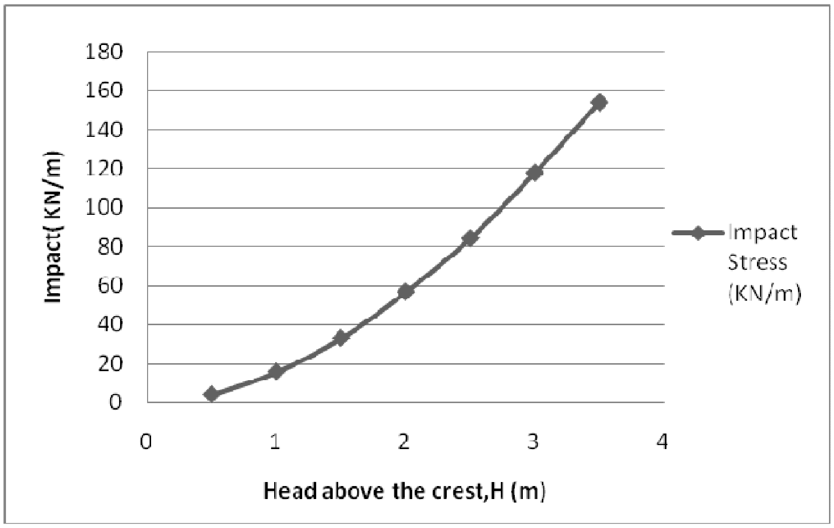

Figure 7: Water impact stress at the vicinity of C/S toe of the crest

The physical processes governing the embankment erosion are closely related to flow-induced local velocity and effective shear stress adjacent to the embankment surface. Equation (6-8) can be used to estimate erosion rate and local critical velocity at incipient condition. All the hydraulic parameters that have been presented relate the head over the crest, depths along the sloping side and tail water depth. Although all of these variables are highly non uniform and varied flow but for sloping part only uniform flow has been considered. Another complicating factor is the change of hydraulic conditions over time as erosion of embankment occurs. Experimental investigation needs to be conducted to get useful data to evaluate the other unknown governing factors for the design of sustainable embankment structure.

5. CONCLUSIONS

An approach has been demonstrated to calculate the hydraulic parameters for flows over a typical coastal embankment. As input to the procedure, sea side water level with extreme low and extreme high magnitudes range between 0.50 m and 3.5 m have been considered for subsequent hydraulic calculations. Flow intensity ranges between 0.40 m^3/s and 14.5 m^3/s has been obtained using the conventional weir formula. Hydraulic parameters e.g. flow velocity, shear stress, sequent depth ratio of hydraulic jumps at toe and impact force of water at the vicinity of toe have been estimated. Analysis reveals that, there have been tremendous water forces of about 3.95 KN/m to 153.94 KN/m acting on the toe when the corresponding head above crest is 0.5 m to 3.5 m respectively. Erosive forces begin at the toe and propagate towards the sloping side causing sudden damage to the structure due to such water impact. Thus the present analyses emphasis the need of necessary erosion resistance measures (soft or hard) for sustainable design of countryside embankment slope undergoes flood or storm surge overtopping.

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