BENEFIT FROM LIQUEFACTION REMEDIAL MEASURES

Dr.Swapan Kumar Bagui*[1], Md. Joynul Abadin[2]

[1]*HOD(Pavement, Material & Geotechnical Division), Intercontinental Consultants & Technocrat Pvt. Ltd, India , e-mail: swapanbagui@gmail.com*

[2]*Executive Engineer, RHD , Bangladesh, e-mail: joynul.rhd@gmail.com*

***Corresponding Author**

ABSTRACT

Liquefaction is a geotechnical problem / hazard in foundation engineering. It increases the pile length due to decreasing vertical pile load carrying capacity and lateral pile capacity and number of pile requirement shall be more at abutment location. Open foundation resting on liquefied layer is also a risk and it shall be avoided as mentioned AASHTO LRFD 2020. This paper presents a case study for foundation analysis and assesses the benefit, if remedial measures of liquefaction is carried out. It is observed from Geotechnical Report that 9 m is liquefied depth. There are several remedial methods for ground improvement against liquefaction. In the present study, ground improvement is in the form of sand column adopted and viability of ground improvement is also determined. It is found that individual pile length was 38.5 m for pile diameter of 1.2 m for liquefaction case and length of pile has been reduced to 34 m i.e., 11.7 % saving in pile length. Number of Pile at abutment location is also reduced. This saving in pile construction cost can be compared with ground improvement cost and ground improvement proposal can be adopted for pile design. This paper also predicts the changes in SPT value for the case of open foundation in cohesion less soil and determines improved bearing capacity of open foundation theoretically.

Keywords: *Liquefaction, Ground Improvement, Sand Compaction Column, Vibro Stone Columns*

1. INTRODUCTION

Liquefaction happens when pore water pressure at a depth of cohesion-less soil is same to its total pressure i.e., the net pressure /stress is zero. As a result, the soil fails its resistance and causes large distortion and even failure of superstructures and substructures. Liquefaction often happens when cohesion-less soil is soaked, loose, and subjected to Seismic loading. To determine liquefaction risk, the seismic-induced extreme shear stress/pressure in soil needs to be estimated.

Figure 1: presents an unbending soil pillar with a unit cross-sectional subjected to ground quickening.
The shear stress is τ = Fe / A = ma/1 = Wa/g = γz a/g = $\sigma z_0 a_{max}$ / g (1)
Where;
Fe = Seismic-induced force;
A = Sectional area of the pillar (A = 1);
m = mass of the Pillar;
W = weight of the pillar;
γ = total unit weight of the pillar;
z = height of the pillar;
a = ground acceleration;
a_{max} = maximum horizontal acceleration at surface due to earthquake;
g = gravitational quickening;
σz_0 =Burden strength at z.

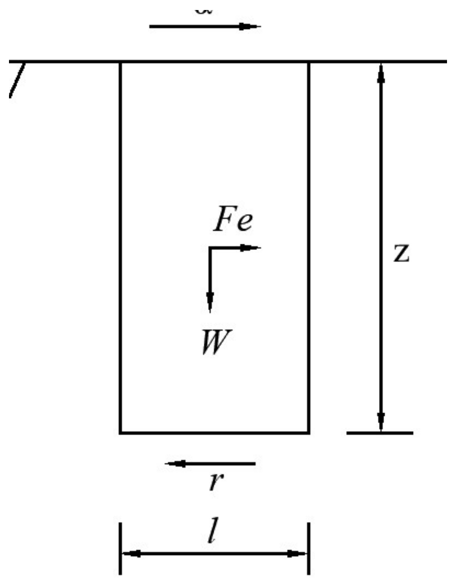


Figure 1: Maximum Shear Stress at the Base of a Rigid Wall

The occurrence of soil liquefaction consists of tough shaking ground wave, relatively free cohesion-less soil and lack of drainage at the time of the seismic activity which causes to surplus pore water pressure development and reducing in effective pressure. It is true that preventing of liquefaction, or reducing its harmful belongings, would consist of increasing density of the soil deposit or the creation of drainage paths accomplished of dispersing pore water pressures /stress more quickly than these are created. Other alternatives to decrease the probable for soil liquefaction consist of growing the narrowing pressure and strengthening that indicates to a lessening of shear stresses supported by the soil (Hayden and Baez 1994).

Risk of liquefaction may be reduced by increasing density of soil by dynamic compaction /vibro compaction, improving drainage path and reducing settlement. Lateral deformation caused by earthquake activities. Mitchell et al. 1995 predicted thirty case studies for which seismic presentation results are available for locations where soil modification was adopted. These consisted of project type, techniques of ground modifications, pre and post-ground usage soil conditions, shaking features, and the effect of the seismic on modified and unmodified condition. They concluded that ground modification can improve protection against liquefaction and ground distress and that disbursements and horizontal spreading can be reduced to acceptable levels. Their modification methods were applied at one hundred seventy five sites, modification or hardening was adopted at sixteen sites, drainage proposal to reduce saturation was used at twenty two locations, and drainages to stop excessive build-up of pore pressures were used at one hundred one sites in 1994. Hayden and Baez (1994) recorded ninety three projects in North America at which ground modifications for liquefaction remedial measures were adopted.

Failure of superstructure resting on open / shallow foundation is one of the most catastrophic phenomena occurring due to liquefaction during earthquake. The benchmark model simulation has been simulated first to obtain the dynamic behaviour of a loose sand deposit with a surface footing. The responses of this model treated with stone column improvement under the same seismic loading has been analysed and compared with the response of Benchmark Models (BM), focusing on the evaluation of the strengthening effect of soil columns and its effect on the behaviour of the remediated soil deposits. Acceleration base input excitation of El Centro earthquake is applied to each model to monitor the displacements, liquefaction potential, and Excess Pore Pressures (EPP). Based on the response of the model, the relative effectiveness of stone columns as mitigation measure can be gauged. A significant reduction in EPP and settlement are visible with the use of stone column as remedial measures (Kumari et al. 2018).

Pre-fabricated vertical (wick) drains have been used in conjunction with stone columns to improve treatment effectiveness in sands with high fines content. However, no comparison testing has been performed with and without drains. Despite the presence of wick drains, improvement effectiveness still decreased as the fines content increased. The average increase in SPT $(N_1)60$ was 114% in a zone with an average of 31% fines, but decreased to about 70% in a zone with an average fines content of 43%. Treatment effectiveness was often minimal in layers containing 15% or more clay sized particles. Significant increases in SPT values were observed with time after treatment in the test areas with drains, but little improvement with time was observed in areas without drains. Increased penetration resistance was similar to that observed at other stone column projects where similar soils were encountered, and wick drains were also used (Rollins et al. 2012).

Various techniques currently exist to improve liquefaction susceptible soils during earthquakes, such as densification, cementation, drainage, and replacement (Towhata 2021). Among such mitigation techniques, the injection of low viscosity polymer has shown potential (Traylen et al. 2016), particularly at locations that might otherwise be difficult to treat with conventional techniques (e.g., under existing buildings and in dense urban environments). A number of earlier laboratory studies demonstrated the beneficial effects of polymer remediation across various soil types (Gatto et al. 2021). In addition to laboratory studies, field trials in Turkey (Erdemgil et al. 2007) reported significant increases in the post injection standard penetration test blow counts (SPT N-value) at sites that suffered extensive damage during the 1999 Kocaeli earthquake.

2. GROUND MODIFICATION TECHNIQUES FOR LIQUEFACTION MITIGATION

Many ground modification techniques are in usage for liquefaction remedial. Vibro compaction systems, compaction, grouting, permeation grouting, and jet grouting are the different ground modification techniques. Mitchell et al. (1995) described that the sites were assessed following the 6.9 magnitude (M) Loma Prieta earthquake were more than 60 km from the epicentre, and the duration was half of that normally expected for an earthquake of this magnitude. These locations were not verified to acceptable limits. It is significant to reminder that the sites generally implemented very well in relative to next to, unmodified areas, many of which faced problems associated with intense ground shaking: liquefaction, sand boils, lateral movement, settlement, and cracking.

2.1 Vibro Systems

Vibro systems can be subdivided into two types: vibro compaction and vibro stone columns (vibro replacement or vibro displacement). These systems use basically the same equipment—a vibrating probe 300 mm to 600 mm in diameter. The probe generates horizontal vibrations that increase the density of the adjacent granular soils. In addition, when a stone backfill is used, the resulting densified column reinforces the soil mass as a higher modulus inclusion. Follower tubes added to the probe allow depths of up to 30 m to be treated.

2.2 Vibro Stone Columns

Vibro stone columns are generally applied to increasing density of soil, creation of drainage path, strengthen, and partially replacement of liquefiable strata. Diameter varied between 7.5 cm and 100.0 cm by adding aggregate in replacement of cohesion-less soil around the vibrator. Stone columns are more permeable than the surrounding soil. This helps in the indulgence of pore water during an earthquake time. Typical friction angle of stone column varies from 38^0 and 45^0; therefore, the vibro stone column adds a reinforcement element with shear strengths greater than the surrounding soils. A method of analysis for vibro column was presented by Baez and Martin (1993).

2.3 Compaction Grouting

In compaction grouting, mortar type grout with slump less that 7.5 cm is pumped into ground under pressure of typically, 0.690 MPa to 2.760 MPa. This method increases the density of surrounding soils by hollow movement moralities. The method is used where access is difficult, either from perpendicular or horizontal allowance, or to strengthen soils below present structures. It is also useful if a soil stratum at depth is the only one requiring modification. It needs overburden pressures in excess of 2.5 m of soil so that movement occurs in the horizontal direction.

2.4 Rapid impact compaction

This is a transitional compaction method between conventional light compaction and deep dynamic compaction. It increases the density geometrical by constantly dropping a hydraulic hammer mounted on an excavator at a rapid rate. The mass of knock is typically 5.0–12.0 Ton which is allowed to fall without obstruction from a height of 1.2 m on a rounded steel foot with a radius of around 0.75m. Typical improvement of SPT is shown in Table 1.

Table 1: Rapid Impact Compaction

Soil Type	SPT after Improvement	Modification Depth(m)
Cohesion-less soil	20-30	6.0
Silty Sand(SM)	15	3.0
Sandy Silt	10-15	3.3 -5.0
Abandoned Fill	>10	3.0 to 5.0

3. PROPOSED METHODOLGY CONTROL LIQUEFACTION

Ground modification is a method to reduce the presence of liquefaction by modifying shear strength parameters by increasing the density of the soil below ground.

Growing effective stress is a technique to regulate the presence of liquefaction by aggregate the effective stress in the soil to avoid elevation of the excess pore water stress / pressure ratio.

Distribution of excess pore water pressure is another method to fight the manifestation of liquefaction by using materials with high permeability in the ground to diffuse excess pore pressure rapidly which is the cause an earthquake.

Monitoring shear distress is a method to decrease the existence of liquefaction by developing better structures with high shear rigidity to reduce shear buckling occurs in the ground during earthquake condition.

3.1 Sand Compaction Pile Method

Sand Compaction Pile (SCP) Method is a technique for ground improvement / modification. This method is useful for both soft soil and loose soil. In this method, steel pipes are bored into the ground, coarse , medium sand is inserted into the pipes, and sand piles are formed by shaking compaction in the ground. It grips cohesion-less soil ground with vibration to densify the surrounding ground and clay soil ground with sand piles to allocate stress and speed up drainage. The major modification of the SCP method consists of increasing the bearing capacity, reducing settlement, avoiding liquefaction by increasing SPT. It also increases horizontal resistance for loose cohesion-less soil/ geo-material ground /cohesive soil ground. Sand, gravel, Recycled Asphalt Concrete (RAP) and improved sand may be used. These processes, termed as gravel compaction piling method which works the same modification principles as the SCP Method.

The technique and scheme differ between the application of the method to sandy soil ground and to cohesive soil ground. Coarse sand is easily available in Bangladesh and sand compaction pile may be adopted for ground improvement against liquefaction and reducing settlement of loose sand.

3.2 Design of Sand Compaction Pile

The design method for modification using the SCP for cohesion-less soil ground includes setting the N-value before and modification .This can be calculated by the sand compaction pile replacement ratio A_S that satisfies this N-value. It is based on reducing the void ratio by sand piles as shown in **Figure 2**. It is assumed that void ratio of the initial soil as e_0 and the void ratio after modification is e_1. The change in void ratio is defined by $\Delta e = (e_0 - e_1)$ which are inoculated into a ground measurements of $1+ e_0$ and the ground is densified. The replacement proportion is as given by **Equation. 2:**

$$A_S = (e_0 - e_1)/(1+ e_0) \tag{2}$$

In SCP calculation, the void ratio can be estimated from the N-value through the relative density Dr. The N-value after modification is mainly compared by the initial N-value before modification (N_0) and the replacement ratio A_S.

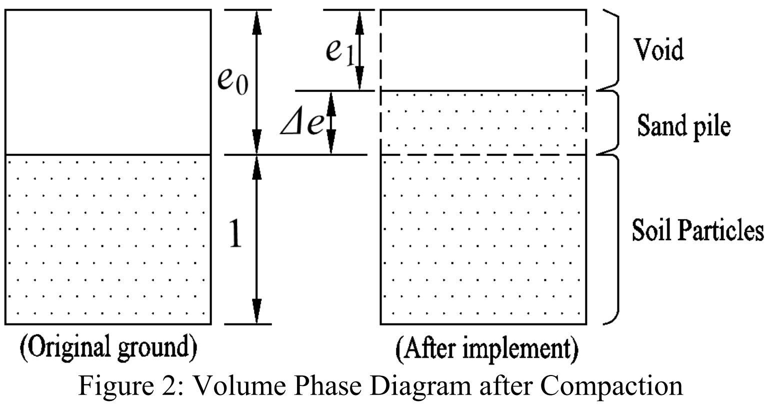


Figure 2: Volume Phase Diagram after Compaction

Based on **Priebe's Methodoloy (1998)**, the shaft can never be unsuccessful in pile tip resistance and any deformation of loaded area results in a deformation of the shaft which remains same all over the length of the shaft. Modification of a soil attained at these conditions by the providing of stone/sand shaft. Furthermore, the soil adjacent to the shaft is assumed to be moved during the stone shaft putting in place to such an extent that its original resistance corresponds to the liquid state and K value is equal to 1. The modification factor n_0 defined as:

$$n_0 = 1 + \frac{A_c}{A}\left[\frac{5 - \frac{A_c}{A}}{4K_{ac}\left(1 - \frac{A_c}{A}\right)} - 1\right]$$

$$K_{ac} = \tan^2\left(45 - \frac{\varphi_c}{2}\right)$$

Where,
n_0=Settlement modification factor
A= Area of the unit cell;
Ac= Area of sand /stone shaft;
Øc= Friction angle of stone or sand, and
A_S=Ac/A.
Relative density can be determined using following Equation:
$Dr=(e_{max}-e)/(e_{max}-e_{min})$ (3)

Maximum and minimum voids can be determined from **Figure 3** based on fine content**.**

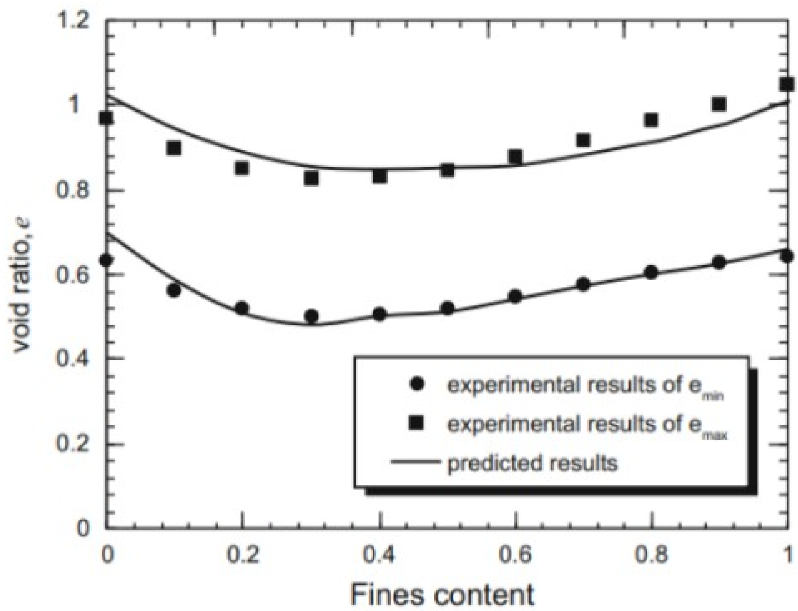


Figure 3: Adopted Maximum and Minimum Void Ratio Based on Fine Content (Source:Shen et al. 2018)

Sand has been classified as very loose to very dense based on I S:6403, I S:2911 Part 1 Sec 2:2010. Relative density, void ratio can be determined based on Annex C of I S:2911, Part 1 Sec 2, Table 3 of IS 6403, Fig.1 and Table 3 of I S :6403. A combined table has been formed for analysis and presented in **Table 2**.

Table 2: Properties of Sand

Soil Type	Field SPT	Void Ratio	Relative Density
Very Loose Sand	0-4	>0.75	< 20 %
Loose Sand	4-10		
Medium Sand	10-35	0.75 -0.55	(20 – 70) %
Dense Sand	35-50	<0.55	> 70%
Very Dense Sand	>50		

4. CASE STUDIES

A case study of Construction of a Bridge is adopted in the present case. The road will be constructed from two lanes to four lane configuration. Geotechnical investigation was carried out at four locations of Bridge at km 26+618. The bridge is located in Seismic Zone II. Bore hole at abutment 1 is considered for the present case study. Bore log data is shown in **Figure 4.**

SAMPLE INDEX			Disturbed				Undisturbed				Soil Classification; ASTM D-2487 & D-2488
Depth(m)	Sample No.	Sample Type	RL	Layer Thickness	Description of Strata Materials	Graphic Log	Blow Count 6"	6"	6"	SPT (N)	Remarks
0.0											
1.5	D-1		6.9	1.5	Rubbish		0	0	0	0	
3.0	D-2		5.4	3.0	Filling Sand, SM		2	5	5	10	
4.5	D-3		3.9				4	6	7	13	
6.0	D-4		2.4	3.0	Grayish Soft to Stiff Clayey Slit, ML		2	4	5	9	
7.5	D-5		0.9				1	1	2	3	
9.0	D-6		-0.6	13.5	Grayish Medium Dense to Dense Fine Silty Sand, SM		5	7	8	15	
10.5	D-7		-2.1				9	10	13	23	
12.0	D-8		-3.6				6	7	9	16	
13.5	D-9		-5.1				6	10	12	22	
15.0	D-10		-6.6				6	11	23	34	
16.5	D-11		-8.1				8	13	19	32	
18.0	D-12		-9.6				9	21	23	44	
19.5	D-13		-11.1				9	11	14	25	
21.0	D-14		-12.6				9	16	18	34	
22.5	D-15		-14.1	9.0	Grayish Soft to Medium Stiff Clayey Silt, ML		1	1	2	3	
	UD-1										
24.0	D-16		-15.6				1	2	2	4	
25.5	D-17		-17.1				1	1	2	3	
27.0	D-18		-18.6				2	2	3	5	
28.5	D-19		-20.1				1	2	4	6	
30.0	D-20		-21.6				1	2	3	5	

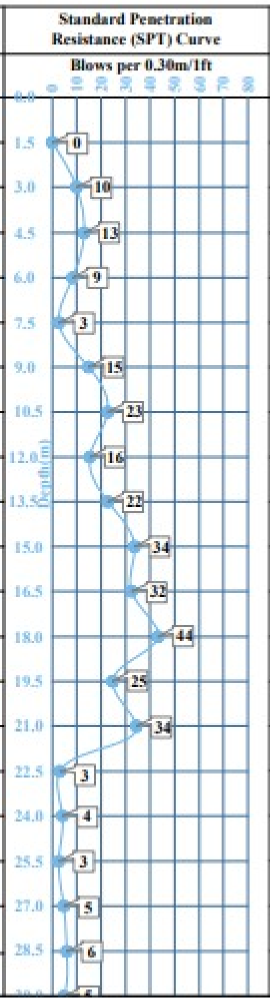
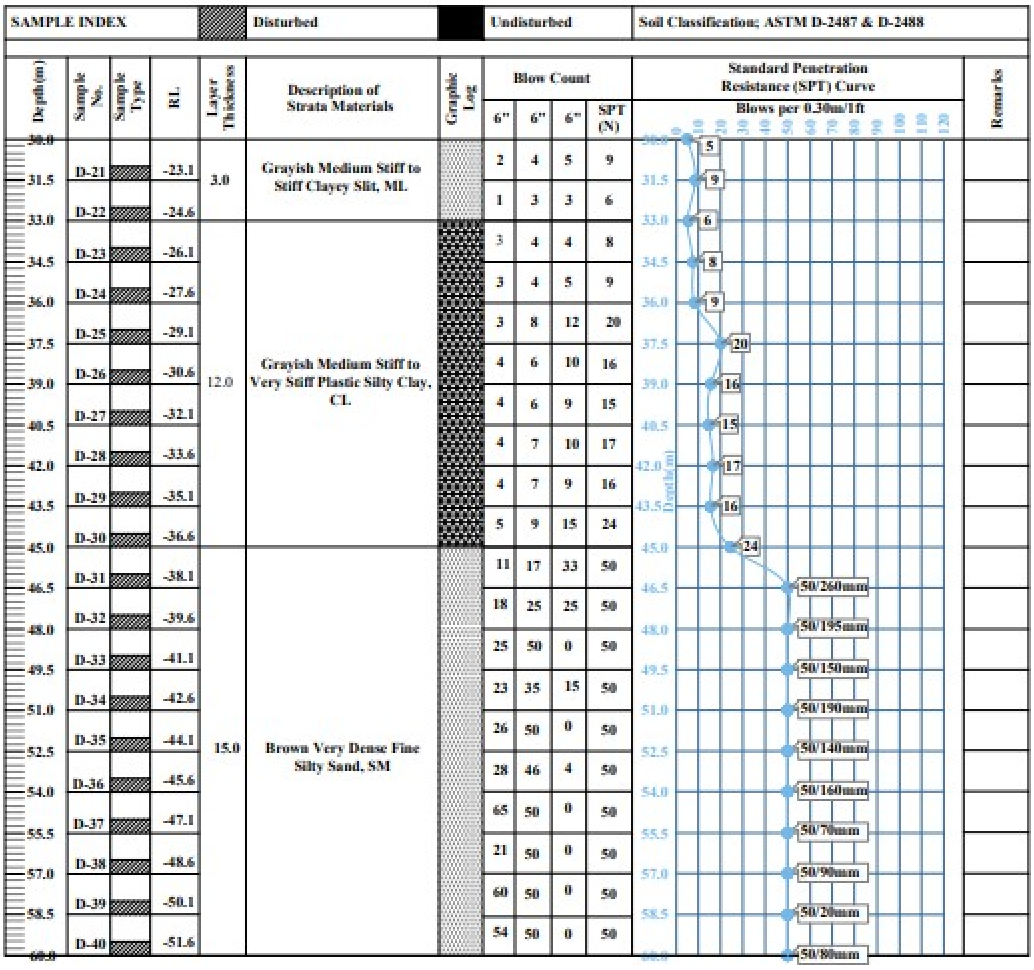


Figure 4 Bore Log Sheet (Page 1 of 2)



Depth (m)	Sample No.	Sample Type	R.L.	Layer Thickness	Description of Strata Materials	Blow Count 6"	6"	6"	SPT (N)
31.5	D-21	Disturbed	-23.1	3.0	Grayish Medium Stiff to Stiff Clayey Slit, ML	2	4	5	9
33.0	D-22	Disturbed	-24.6			1	3	3	6
34.5	D-23	Disturbed	-26.1	12.0	Grayish Medium Stiff to Very Stiff Plastic Silty Clay, CL	3	4	4	8
36.0	D-24	Disturbed	-27.6			3	4	5	9
37.5	D-25	Disturbed	-29.1			3	8	12	20
39.0	D-26	Disturbed	-30.6			4	6	10	16
40.5	D-27	Disturbed	-32.1			4	6	9	15
42.0	D-28	Disturbed	-33.6			4	7	10	17
43.5	D-29	Disturbed	-35.1			4	7	9	16
45.0	D-30	Disturbed	-36.6			5	9	15	24
46.5	D-31	Disturbed	-38.1	15.0	Brown Very Dense Fine Silty Sand, SM	11	17	33	50
48.0	D-32	Disturbed	-39.6			18	25	25	50
49.5	D-33	Disturbed	-41.1			25	50	0	50
51.0	D-34	Disturbed	-42.6			23	35	15	50
52.5	D-35	Disturbed	-44.1			26	50	0	50
54.0	D-36	Disturbed	-45.6			28	46	4	50
55.5	D-37	Disturbed	-47.1			65	50	0	50
57.0	D-38	Disturbed	-48.6			21	50	0	50
58.5	D-39	Disturbed	-50.1			60	50	0	50
60.0	D-40	Disturbed	-51.6			54	50	0	50

Figure 5 Bore Log Sheet (Page 2 of 2)

Liquefaction analysis has been carried out as per guideline of IRC :75-2015 and IRC: SP 114-2019, Liquefaction depth for this bore hole is found to be 9 m. Detail calculation is shown in **Annexure 1.** Vertical capacity and lateral capacity are calculated and both capacities are reduced due to higher liquefaction depths. Detail calculation is carried out based on AASHTO LRFD 2020 for vertical pile capacity and IRC: 78 -2014 for lateral pile capacity and these are presented in Table 3 and **Annexure 1.**

4.1 Improvement Proposal

Sand column is proposed due to availability of coarse sand locally. Assuming diameter of sand column 0.6 m and sand column will be provided @1.5 m interval.
Replacement ratio = As = $(0.6/1.5)^2 = 0.16$.
Average SPT up to liquefaction depth = 9.2 = 9 (say). Sand relative density falls under loose category as shown in Table 2.
Void ratio for calculation purposes can be taken from Figure 3 based on fine content. Average fine content of the liquefied layers were calculated 40 %. Void values are 0.5 and 0.86. Void Ratio, e for SPT 9 is found to be 0.78 i.e., in loose state.

Original void ratio can be determined from Equation 2 and found to be 0.78.
Replacement ratio, $A_S = (e_0 - e_1)/(1 + e_0)$ i.e., $16=(0.86-e_1)/1.86$, Hence, $e_1=0.86-0.16\times1.86=0.0.564$ i.e., void after compaction, it will be medium dense state, since $e_1=0.564$ i.e., SPT = 34(it will be between 30 and 50). Taking SPT Value is 34. Priebe settlement improvement factor has been determined and found to be 1.82. Density will also be increased. It is generally found that SPT after improvement lies between two times and five times of the original values. Hence, from conservative

consideration, it is taken two times i.e., 18. Liquefaction analysis has been revised and found to be that the entire depth of the bore log is non-liquefiable. Vertical and lateral pile capacities are determined after ground improvement and presented in Table 3. Pile length has been re-calculated and it is found that pile length is reduced to 34 m in place of 38.5 m i.e., 11.7 % saving in pile length.

Table 3: Design Summary of the Pile

Case	Plie Length Required(m)	Vertical Capacity Obtained(T)	Lateral Capacity (T)	Depth of Fixity(m)
Without Improvement	38.5	322.0	22	16.6
With Improvement	34.0	320.5	54	9.62

Note: Pile Diameter 1.2 m , Vertical Capacity Required:320T

4.2 Remedial Measure

The factor of safety was calculated and found to be in the range of 0.30 - 0.99 for the liquefaction section as shown in Annexure 1. The different improvement options were discussed and finally, after visiting the site along with constraints and material availability, it was finalised to use sand columns which are available locally. The diameter and spacing of sand column were finalised to mitigate liquefaction potential. This was also verified using Priebe's Method (1998). Sand columns of 600 mm diameter with 1.5 m centre to centre spacing was decided to mitigate the liquefaction potential risk of present site and thereby factor of safety against liquefaction values enhanced more than 1.0.

In order to increase capacities, ground improvement proposal have been adopted. Vertical pile capacity analysis and lateral pile capacity are calculated using AASHTO LRFD 2020 and IRC:78-2014 respectively and summarised results are presented in Table 3. SPT along with density are the major input for the determination vertical capacity using AASHTO LRFD 2020.

5. DISCUSSION

The analysis has been carried out for pile design at liquefied location. Pile is designed without considering ground improvement at liquefaction area. Pile length increases at pier location and number pile increases at abutment location due to decrease of lateral pile capacity.

The analysis has been carried out without and with improvement of the liquefaction location. It is also found from Table 3 that Pile length reduced to 34 m in place of 38.5 m i.e., 11.7 % saving in pile length. Similarly number of pile will be reduced due to increasing lateral pile capacity at the abutment location. Quantity of steel will be reduced due to decreasing depth of fixity from 16.6 m to 9.6 m. This economic benefit can be justified for providing ground improvement. Failure of structures during earthquake can be avoided. This aspect can be considered during the design stage of the structures.

6. CONCLUSIONS

Earthquakes causes damage of structures during soil liquefaction. The purpose of an effective earthquake ground modification proposal is to reduce liquefaction or limit lateral movement. To achieve this, the soil shall be compacted, drainage arrangement provided, strengthen, or replaced. In addition to the methodologies presented in this paper, many ground modification techniques are available: dynamic compaction, stone column, sand compaction pile, vibro systems, compaction grouting, and permeation grouting can be used.

To drain the soil stone columns, PVD, high-capacity gravel drains and permanent dewatering may be used.

Based on the present case study following limited conclusions may be drawn:

- Foundation at liquefied location needs special care to avoid future damages of the structures and embankment;

- A cost comparison may be carried out with and without ground improvement cases. Viable proposals shall be adopted for implementation.
- The different remedial measures including uses of local materials (sand, aggregate, recycle materials) may be considered to check viable option.
- It is recommended to adopt ground improvement at liquefied location.
- Sand Compaction Pile design methodology as mentioned in the present paper may be followed for the case of ground improvement of soft soil /loose soil in the form sand, silty sand and sandy silt soil.
- SPT test and Plate load test shall be conducted before and after constructing ground improvement proposal to check ground improvement and verify theoretical anlysis.
- Similar ground improvement proposal may be adopted for constructing open foundation liquefied ground profile.

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Borehole Diameter (mm)	Sampler used was with liner	Magnitude of Earthquake	Hammer Energy (70% for rope pully system)	Depth below EGL, z (m)	Type of Strata	Observed SPT Value (N_m)	Saturated density (t/m^3)	Submerged Density (t/m^3)	Fine Content (%)	Stress reduction coefficient (r_d)	Total overburden pressure (σ_o), t/m^2	Effective overburden (σ'_o), t/m^2	Cyclic Stress ratio (CSR)	C_N	C_E	C_B	C_R	C_S	SPT corrected $(N1)_{60}$	α	β	$(N_1)_{60cs}$	$CRR_{M=7.5}$	CRR	FOS	Conclusion	NL=Non-Liquefiable, L=Liquefiable
150	Yes	6.5	70	1.5	Silt/Sand	0	1.6	0.6	98	0.99	2.4	0.90	0.34	1.70	1.17	1.05	0.75	1	0.00	5.00	1.2	5.00	0.07	0.10	0.30	L	
150	Yes	6.5	70	3	Silt/Sand	10	1.68	0.68	1.8	0.98	4.92	1.92	0.33	1.70	1.17	1.05	0.80	1	17.00	0	1	17.00	0.18	0.26	0.80	L	
150	Yes	6.5	70	4.5	Silt/Sand	13	1.75	0.75	1.8	0.97	7.545	3.05	0.31	1.70	1.17	1.05	0.85	1	22.10	0	1	22.10	0.24	0.35	1.13	NL	
150	Yes	6.5	70	6	Silt/Sand	9	1.65	0.65	98	0.95	10.02	4.02	0.31	1.58	1.17	1.05	0.95	1	14.19	5.00	1.2	22.03	0.24	0.35	1.13	NL	
150	Yes	6.5	70	7.5	Silt/Sand	3	1.62	0.62	98	0.94	12.45	4.95	0.31	1.42	1.17	1.05	0.95	1	4.26	5.00	1.2	10.12	0.11	0.16	0.53	L	
150	Yes	6.5	70	9	Silt/Sand	15	1.75	0.75	1.81	0.93	15.075	6.08	0.30	1.28	1.17	1.05	0.95	1	19.25	0	1	19.25	0.21	0.30	0.99	L	
150	Yes	6.5	70	10.5	Silt/Sand	23	1.85	0.85	1.81	0.89	17.85	7.35	0.28	1.17	1.17	1.05	1	1	26.83	0	1	26.83	0.33	0.48	1.70	NL	
150	Yes	6.5	70	12	Silt/Sand	16	1.75	0.75	8	0.85	20.475	8.48	0.27	1.09	1.17	1.05	1	1	17.38	0.30	1.01	17.90	0.19	0.27	1.02	NL	
150	Yes	6.5	70	13.5	Silt/Sand	22	1.83	0.83	1.81	0.81	23.22	9.72	0.25	1.01	1.17	1.05	1	1	22.31	0	1	22.31	0.25	0.36	1.41	NL	
150	Yes	6.5	70	15	Silt/Sand	34	1.92	0.92	1.81	0.77	26.1	11.10	0.24	0.95	1.17	1.05	1	1	32.27	0	1	32.27	NA	NA	NA	NL	
150	Yes	6.5	70	18	Silt/Sand	32	1.89	0.89	1.81	0.69	31.77	13.77	0.21	0.85	1.17	1.05	1	1	27.27	0	1	27.27	0.35	0.50	2.40	NL	
150	Yes	6.5	70	19.5	Silt/Sand	44	1.99	0.99	1.81	0.65	34.755	15.26	0.19	0.81	1.17	1.05	1	1	35.62	0	1	35.62	NA	NA	NA	NL	

Annexure 1

An Excel sheet has been developed based on design procedure mentioned in AASHTO LRFD 2020 and pile capacity is determined and vertical pile capacity is found to be 322 T for pile diameter of 1.2 m and pile length below pile cut-off of 38.5 m. Lateral pile capacity is found to be 22 T.

Pile capacity is determined and vertical pile capacity is found to be 320.5 T for pile diameter of 1.2 m and pile length below pile cut-off of is 34 m. Lateral pile capacity is found to be 54 T.