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Analysis of Liquefaction and its Mitigation for Soil under Open Foundation of Structure - A Case Study of Road Project in West Bengal, India

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Abstract. Liquefaction of loose to medium dense saturated cohesionless sandy soil deposit endangers the stability of open i.e. shallow foundation of structure under seismic loading condi-tion due to loss of shear strength of underlying liquefiable soil and subsequent occurrence of excessive settlement. This paper presents the typical case study of assessment of liquefaction potential of the loose to medium dense sandy foundation soil deposits having high water tableas found through geotechnical investigations at one ROB structure location along the project road of NH-31D from “Siliguri (Ghoshpukur)” to “Alipurduar (Salsalabari)” in the northernpart of West Bengal. Since the said project road is located in the “Seismic Zone IV” of “IS 1893 (Part 1): 2016 (Reaffirmed 2021)” [1], so there was susceptibility of liquefaction of loose to medium dense saturated cohesionless soil explored under the open foundation of substruc- tures. The structures having shallow foundations were not feasible due to the presence of lique- fiable depth of soil beyond the proposed founding level at those identified locations of substruc- tures. Hence it was essential to do the appropriate mitigation measure against liquefaction of thefoundation soil deposits. The “vibro-probing method” of densification for loose to medium dense saturated sandy soil was adopted as the effective and economic mitigation measure against probable liquefaction in seismic condition. The paper discusses about the said ground treatment method of densification for loose to medium dense saturated sandy deposits under theopen foundations of substructures. The field geotechnical investigation work by SPT method of sounding was conducted at the post-ground treatment stage and the liquefaction potential forthe treated foundation soil was also assessed to verify the efficiency of adopted mitigation measures against any possibility of liquefaction during earthquake.

Keywords: Earthquake; Seismic Loading, Soil Liquefaction Potential; Open Foundation, Miti- gation Measure, Vibro-Probing Method of Densification of Soil.

1. Introduction

The rehabilitation and upgradation work of existing intermediate/2-lane road of NH-31D to 4-lane configuration from “Ghoshpukur” to “Salsalabari” which is around 84 km of length was undertaken by the “National Highways Authority of India (NHAI)”, Govt. of India during last few years starting from middle of 2015. There are various structures namely major bridges, flyovers, Road over Bridges (ROB), minor bridges and culverts along the project road for which the detailed geotechnical investigations were conducted to explore and characterize the engineering properties of foundation soil at the respective locations of structures. The broad nature of foundation soil at the various structure locations along the project corridor is found as non-plastic (np) sandy soil of varying densities along with intermittent patch of low plastic silly/clayeysoils. Since the project road is situated in “Seismic Zone IV” of “IS 1893 (Part 1): 2016 (Reaffirmed 2021)” [1] and the ground water table is also close to the exiting ground level, so there are possibilities of occurrences of “Liquefaction” of the loose tomedium dense non-plastic saturated sandy soils which were present at shallow to moderate depth of foundations of structures under seismic condition.

The case study of engineering assessment of “Liquefaction Potential” including the “Liquefaction induced Settlement” for the foundation soil layers of one “Road over Bridge (ROB)” structure location along the project road having shallow (i.e. open) foundation is presented at the initial part of this paper. The later part of the paper de- scribes about the adopted ground treatment measure namely densification of the loose to medium dense silty sand/sand by the application “vibro-probing method” till up to the desired depth for ensuring the stability of the shallow foundation of the viaduct portion of the ROB structure under seismic condition. The detailed assessment of the “Liquefaction Potential” of the foundation soil layers at post-treatment stage which was done by using the investigation records as obtained through “Standard Penetra- tion Test (SPT)” at the shallow foundation locations of the ROB structure is coveredat the end part of the paper.

2. Geotechnical Investigation and Subsurface Stratifications

A comprehensive geotechnical exploration work covering detailed field investigation and laboratory testing was carried out at each and every pier and abutments of the ROB structure for the characterization and detailed assessment of the subsurface con- ditions. The field investigation work was comprised of soil borings, performing in-situ tests namely “Standard Penetration Test (SPT)”, obtaining and preserving undis- turbed (UDS) and disturbed (DS/SPT) soil samples, locating position of ground water table (GWT) and observations of the subsurface conditions. The laboratory program included the detailed testing of all the index, strength and consolidation properties for the various types of soil samples namely UDS and DS/SPT samples as collected from the exploratory boreholes to characterize the geological and geotechnical properties ofthe foundation layer profiles. The field investigation and laboratory testing works were performed as per provision of the latest version of related guidelines like “Bu- reau of Indian Standards (BIS)”. The exploratory boreholes at the pier and abutments locations of the ROB structure were terminated at depth 20 to 30 m by extending inside “Very Dense Silty Sand / Sand mixed with some Gravel Layer”. The general geology of the area, position of ground water table (GWT) as encountered and site specific subsurface conditions are described in the subsequent sections briefly.

2.1 Regional Geology of Project Area

The project site lies in the state of West Bengal which stretches from the "Himalayas" in the north and "Bay of Bengal" in the south. The project road mainly traverses through two major districts of "West Bengal" namely "Darjeeling" and "Jalpaiguri". The greater part consists of detrital and alluvial plains. The general elevation of the land is 80 to 100 meters with some marginal slant.

Geologically the area is occupied by unconsolidated alluvial sediments of quaternary age confined to high level terraces and piedmont plains, unaltered semi-consolidated sediments of the Permian and Tertiary age in the foothills. The Quaternary alluvium is laid down by the South-flowing mountainous streams and rivers. The alluvial deposits in this district have been laid down as flood plains deposits by the torrential mountainous streams and rivers. The sediments comprise boulder, pebbles, gravels and coarse to medium sand intercalated with lenses of clay. The sediments in the northern part of the district are poorly sorted, but the assortment improves slightly towards the south. The boulders (of size cm or more), pebbles and gravels are well rounded and are derived mainly from the Precambrian quartzite, granites, etc. The sands are coarse to fine, sub-rounded and micaceous (muscovite).

2.2 Seismic Condition of the Project Road

The site is located in "Seismic Zone IV" as per "Seismic Zoning map of India given in "IS 1893 (Part 1):2016 (Reaffirmed 2021)" [1]. The said region of "West Bengal" in India has experienced earthquakes at relatively low frequency of the seismic hazard zonation map. In past this region has been subject to earthquakes with 4.5 - 6.1 magnitude. For the analysis of "Liquefaction Susceptibility" of the ROB and other structure locations along project road, the maximum magnitude (Mw) of probable earthquake is considered as 6.50.

2.3 Location of Ground Water Table (GWT)

The "Ground Water Table (GWT)" as observed in the various explored boreholes at the ROB structure was varying from 2.65 m to 4.15 m below the existing ground level (EGL) i.e. bed level of substructure (piers and abutments) locations. For designing the substructures' foundations including doing the assessment of "Liquefaction Susceptibility" of the subsurface soil layers, the critical condition corresponding to the highest position of water level which may occur especially during and immediately after rainy season is considered at the EGL of the respective locations.

2.4 Subsurface Stratifications

The site-specific sub-surface conditions were characterized based on the data obtained through exploratory field borehole investigations and various laboratory tests of the collected soil samples which were conducted under geotechnical investigation works. The foundation soil layer profiles as explored through the boreholes at the respective locations of substructures consisted of following major types of strata along the ROB structure.

- Layer I: Loose to Medium Dense non-plastic (NP) Silty Sand (SM) / Sand (SP) of variable thickness say 7.00 to 10.00 m,

- Layer II: Medium Dense/Dense to Very Dense non-plastic (NP) Silty Sand (SM) / Sand (SP/SW) of variable thickness say 10.00 to 25.00 m

The presence of various subsurface layers, its BIS classification and thickness including the range of engineering properties as explored namely field SPT (N) values (blows/300 mm penetration), Grain Size Analysis (Gravel/Sand/Silt/Clay), Atterberg's Limits (Liquid Limit i.e. LL, Plastic Limit i.e. PL and Plasticity Index i.e. PI), Natural Moisture Content (NMC) and Specific Gravity (Gs) for some of the boreholes as considered at the respective substructure locations along the ROB structure are given below in Table 1.

Table 1. Range of Engineering Properties of Subsurface Layers at Boreholes of "**ROB 1**"

Layer No.	Thickness of Layer (m)		Type of Soil (as per IS)	Field SPT (N) (blows/300 mm penetration)	Grain Size Analysis (Gravel / Sand / Silt / Clay) (%)	Atterberg's Limits (LL/PL/PI) (%)	NMC (%)	Specific Gravity (Gs)
	From	To						
BH-ROB-1/8 (For Pier, P10)								
Layer 1	0.00	8.50	SW-SM/SP	7-26	0-26/66-88/3-34*	NP	19-24	2.65-2.67
Layer 2	8.50	20.10	SM/SW	31-100	5-29/70-86/1-9*	NP	21	2.65
BH-ROB-1/12 (For Pier, P14)								
Layer 1	0.00	8.50	SM/SP	1-21	0-1/81-99/1-18*	NP	11-19	2.62-2.64
Layer 2	8.50	10.00	GP	23	61/37/2*	NP	19	2.63
Layer 3	10.00	35.28	SP	69-100	0-2/97-99/0-2	NP	20-23	2.62-2.66
BH-ROB-1/14 (For Pier, P15 & P16)								
Layer 1	0.00	8.50	SP	2-12	0/97-99/1-3*	NP	15-16	2.65-2.66
Layer 2	8.50	10.00	GP	30	79/20/1	NP	16	2.72
Layer 3	10.00	20.20	SP	35-100	0-15/85-99/0-1*	NP	19-21	2.66
BH-ROB-1/17 (For Pier, P18 & P19)								
Layer 1	0.00	10.00	SP	2-22	0-7/90-99/1-3*	NP	12-17	2.62-2.64
Layer 2	10.00	20.29	SP/SM	43-100	0/92-99/1-8*	NP	19-21	2.64-2.66
BH-ROB-1/18 (For Pier, P20)								
Layer 1	0.00	7.00	SP	4-13	7-24/74-91/2*	NP	12-14	2.67-2.68
Layer 2	7.00	20.23	SP/SW	35-100	0-30/66-100/0-4*	NP	16-21	2.63-2.70
BH-ROB-1/19 (For Pier, P21)								

Layer No.	Thickness of Layer (m)		Type of Soil (as per IS)	Field SPT (N) (blows/300 mm penetration)	Grain Size Analysis (Gravel / Sand / Silt / Clay) (%)	Atterberg's Limits (LL/PL/PI) (%)	NMC (%)	Specific Gravity (Gs)
	From	To						
Layer 1	0.00	10.00	SP	1-25	0-2/95-98/2-3*	NP	15-16	2.64-2.65
Layer 2	10.00	20.11	SP/SW	59-100	2-28/88-97/1-4*	NP	18-21	2.65-2.66
BH-ROB-1/21 (For Pier, P23)								
Layer 1	0.00	10.00	SP	6-25	0/99-100/0-1*	NP	9-15	2.60-2.63
Layer 2	10.00	20.40	SP	57-100	0/99/1*	NP	14-20	2.62-2.72
BH-ROB-1/22 (For Pier, P24)								
Layer 1	0.00	4.00	SP	3-9	0/99/1*	NP	12	2.65
Layer 2	4.00	11.50	SP	11-44	36/62/2*	NP	18	2.69
Layer 3	11.50	35.35	SP	51-100	0/97-99/1-3*	NP	22-23	2.59-2.65
BH-ROB-1/23 (For Pier, P25)								
Layer 1	0.00	10.00	SP	3-26	1-6/93-99/0-1*	NP	18-19	2.60-2.63
Layer 2	10.00	20.22	SP	62-100	0/99/1*	NP	21-22	2.63-2.65
BH-ROB-1/24 (For Abutment, A2)								
Layer 1	0.00	10.00	SP	4-21	0-1/96-99/1-4*	NP	15-17	2.64-2.68
Layer 2	10.00	20.23	SP	42-100	0/97-98/2-3*	NP	20-21	2.61-2.63
BH-ROB-1/25 (For Abutment, A2)								
Layer 1	0.00	8.50	SP	3-25	0-4/95-99/1*	NP	11-19	2.64-2.66
Layer 2	8.50	20.45	SP	29-100	0-1/98-99/1*	NP	19-20	2.64-2.71

Note: *Combined portion of "Silt" and "Clay".

From the above table of engineering properties of subsurface layers it can be noted that there are presence of "loose to medium dense non-plastic (NP) silty sand (SM) and / or poorly graded sand (SP)" layers till up to 7.00 to 10.00 m depth from the existing ground level in most of the above stated borehole locations of ROB structure.The top "loose to medium dense silty sand (SM) / poorly graded sand (SP)" layer is underlain by "dense to very dense silty sand (SM) / poorly graded (SP) / well graded (SW) sand" in all the boreholes of the said ROB. The layer of "poorly graded mediumdense to dense gavel (GP)" is found only at the boreholes namely "BH-ROB-1/12"and "BH-ROB-1/14" immediately above the bottom layer of the "dense to very dense silty sand (SM)`/ poorly graded (SP) / well graded (SW) sand".

The above stated engineering properties of the subsoil layers are used for the analysis of “Liquefaction Susceptibility” including the evaluation of “Liquefaction Potential” and the post occurrence effect of “Liquefactions” of the existing subsurface soil layers at the foundation locations of substructures (piers and abutments) of the said ROB.

3. Liquefaction of Soil and its Post Occurrence Effects

3.1 Introduction to Liquefaction of Soil

Soil liquefaction is a phenomenon in which a cohesionless soil deposit below the groundwater table loses a substantial amount of strength due to pore pressure generation resulting from earthquake strong ground shaking. The pore pressure is generated due to the compaction of cohesionless soil during earthquake shaking and this tendency causes the pore water pressures in the soil to increase until the porewater has time to dissipate from the soil skeleton. This pore pressure increase, in turn, causes a reduction in effective stress and associated reductions in soil strength. The saturated soil which loses its strength and stiffness due to earthquake shaking isknown as liquefiable soils. The different types of hazards caused by liquefaction can be grouped into the following categories.

Flow Slides. The most catastrophic ground failure that can be triggered by liquefactions is “Flow Failures”. Large translational or rotational flow failures are produced when the average static (gravity) shearing stresses on potential failure surfaces exceed the low residual strength of the liquefied soil (the static factor of safety drops below 1.00 due to “Liquefaction”). This is usually become critical for any sloping ground or embankment.

Lateral Spreading. This damage has often been associated with translational slides and related embankment deformations due to progressive but limited lateral spreading deformations of the order of feet, driven by earthquake ground shaking subsequent to liquefaction, with deformations ceasing at the end of the earthquake.

Reduction in Bearing Capacity. During Liquefaction, the soil behaves mostly as a liquid and loses its strength so the bearing capacity of the foundation is reduced drastically. It results in the failure of the structures resting on the foundation.

Ground Settlement. The liquefaction induced ground settlement can occur due to the dissipation of excess pore pressure even in the absence of flow sliding, lateral spreading, or reduction in foundation bearing capacity due to liquefaction.

Sand Boils. Sand boils often develop after the occurring of liquefaction. During earthquake shaking, seismically induced excess pore pressures are dissipated predominantly by the upward flow of pore water. This flow produces upward-acting forces on soil particles. If the hydraulic gradient driving the flow reaches a critical value, the vertical effective stress will drop to zero and in result the soil will be in quick condition. In such cases, the water velocities could also be sufficient to transport the soil particles to the surface.

3.2 Factors affecting Liquefaction Susceptibility of Soil

Before doing the evaluation of "Liquefaction" and its effect of any site, the potential susceptibility for "Liquefaction" of the site should be checked. There are different methods for examining the susceptibility of the site for the probable occurrence of "Liquefaction". The following screening criteria are most commonly used to assess the probability of "Liquefaction".

Historical Records. The behavior of liquefaction can be obtained from the post-earthquake field investigations. It is noticed that the recurrence of liquefaction often happens at the same location when soil and groundwater conditions have remained unchanged for long period (Youd, 1984a) [5]. The historical evidences of liquefaction were abundantly used to map the liquefaction susceptibility of any area.

Geologic Age. The soil deposits which are susceptible to liquefaction are generally formed within a relatively narrow range of geological environments (Youd, 1991) [5]. The liquefaction susceptibility of soil depends upon the depositional and hydrological environment and age of the soil (Youd and Hoose, 1977) [5]. The geological formations having mostly uniform grain size distribution in loose deposition usually show high liquefaction susceptibility. Similarly, the fluvial, colluvial and aeolian deposits under saturation are likely to be susceptible to liquefaction. The liquefaction susceptibility of the older deposits is generally lower than that of recent or newly formed deposits (i.e. geologically young).

Composition. Due to the essential requirement of development of excess pore pressure for occurrence of the liquefaction, the susceptibility of liquefaction is majorly influenced by the compositional characteristics of the soil deposit that influence the volume change behavior. The compositional characteristics namely particle size, shape and gradation of soil deposits, which associated with high volume change po- tential, generally tend to be associated with high liquefaction susceptibility. The lique-faction related occurrences were mostly considered to be limited to fine sand. Poorly and/or uniformly graded soil mass are prone to liquefy than well graded soil mass. In the recent past the works of various researchers have established that the gravelly soil can also be found susceptible to "Liquefaction" when the pore pressure dissipation is obstructed due to existence of undrained condition by the presence of any impermea- ble layer within the gravelly soil [5]. Even though the clayey soil is generally not susceptible to liquefaction, however the "strain softening" behavior which is similar to the liquefaction can be observed for the sensitive clay under seismic condition. The liquefaction susceptibility generally reduces with increase in fine contents and "Plasticity Index (PI)" of the soil mass [6] [7]. The fine grained soils which satisfy the following all the "Chinese Criteria (Wang 1979)" can be considered susceptible to significant loss of its strength under appropriate seismic condition [5].

i. Fraction finer than 0.005 mm <= 15%
ii. Liquid Limit, LL <= 35%
iii. Natural Water i.e. Moisture Content (NMC) > 0.90 LL
iv. Liquidity Index <= 0.75.

Saturation. Even though past occurrences of liquefaction are reported for the unsaturated soils, however minimum 80-85% degree of saturation is usually found to be necessary condition for soil liquefaction [6] [7]. For the initial screening of "Liquefaction Susceptibility" at any place, the highest recorded position of the ground water table (GWT) is generally to be considered in case of field recording of ground water table fluctuations with seasonal variations in the place.

State of Soil including Soil Penetrating Resistance. Liquefaction susceptibility is very much dependent on the initial state i.e. relative density/stiffness and void ratio of the soil mass at the time of earthquake. As per the data presented by Seed and Idriss (1982), the liquefaction has been observed for the soil mass having normalized "Standard Penetration Test (SPT)" blow counts i.e. $(N_1)_{60}$ <=22. The threshold value of normalized SPT i.e. $(N_1)_{60}$ above which the "Liquefaction" will generally not occuris given as 30 by "Marcuson" (1990). However, the Chinese experience as reported in Seed, et al. (1983) suggests that the liquefaction is possible to occur in soil mass having normalized SPT blow counts value i.e. $(N_1)_{60}$ till 40 [6] [7].

Seismic Zones. The liquefaction susceptibility of the soil deposit is also dependent on the intensity of vibration generated due to earthquake depending upon the "Seismic Zones". As the intensity of the earthquake increases with increase in the "Seismic Zone" as per the BIS [IS 1893 (Part 1): 2016] guidelines, so the liquefaction suscepti- bility of the soil deposits in the higher seismic zone namely "Zone IV" and "Zone V" are more than the seismic zone like "Zone II" and "Zone III".

If majority of the above stated criteria are satisfied for any soil deposit, then the "Liquefaction Potential" of the deposits shall need to be evaluated. The recommendations on the "Liquefaction Susceptibility" of the soil mass based on its type, state and saturation level as given in various Indian guidelines namely (i) IS 1893 (Part 1) : 2016, (Reaffirmed 2021) [1], Clause No. 6.3.5.3, Indian Standard Criteria for Earth- quake Resistant Design of Structure, Part 1, General Provisions and Buildings (Sixth Revision) by Bureau of Indian Standards (BIS), New Delhi (ii) IRC:SP:114 - 2018 [2], Clause No. 8.4.4, Guidelines for Seismic Design of Road Bridges by Indian Road Congress, New Delhi (iii) IRC:75-2015 [3], Clause No. 3.9, Guidelines for the Design of High Embankments (First Revision) by Indian Road Congress, New Delhi and (iv) Report No. BS-118 [4], Clause No. 13.3, RDSO Guidelines on Seismic Design of Railway Bridges (Version 1.0), November 2015, Ministry of Railways, Govt. of India are presented here below in **Table 2**.

Table 2. Recommendations of Indian Guidelines on "Liquefaction Susceptibility"

IS 1893 (Part 1) : 2016 (Reaffirmed 2021) [1], Clause No. 6.3.5.3	IRC:SP:114 - 2018 [2], Clause No. 8.4.4	IRC:75-2015 [3], Clause No. 3.9	RDSO Report No. BS-118 [4], Clause No. 13.3
i. Liquefaction susceptibility is generally found for the soil deposits consisting of submerged loose sands and soil falling	i. Saturated cohesionless loose sandy soil with or without silty/clay fines are mostly found susceptible to liquefac-	i. Unsaturated soil are not prone to liquefaction ii. Liquefaction generally takes place in loose	i. Loose sand orpoorly graded sand (SP) with lit- tle or no fines in saturation are prone to liquefac-

IS 1893 (Part 1) : 2016 (Reaffirmed 2021) [1], Clause No. 6.3.5.3	IRC:SP:114 - 2018 [2], Clause No. 8.4.4	IRC:75-2015 [3], Clause No. 3.9	RDSO Report No. BS-118 [4], Clause No. 13.3
under classification SP i.e. poorly graded sand with corrected SPT (N) values less than 15 in Seismic Zones III, IV and V, and less than 10 in Seismic Zone II ii. Marine Clay and other Sensitive Clay layers of low shear strength may befound susceptible to liquefaction iii. Soils with corrected SPT (N) value equivalent to clean sand more than 30 i.e. $(N1)_{60CS}$ > 30 are not susceptible to liquefaction ("Annex F" of IS 1893 (Part 1) :2016)	tion ii. Liquefaction susceptibility may not be present for the saturated sandy soil below 20 m depth from OGL iii. Liquefaction hazards may be neglected in case of fulfilling of any one of the following conditions • Sands have clay content >25% and "Plasticity Index (PI)> 10 • Sands have silt content > 35% and corrected N value i.e. $(N1)_{60}$ > 20 • Sands are clean with corrected SPT (N) value more than 30 i.e. $(N1)_{60CS}$ > 30	fine grained sand (having finer than 75 micron <5% and 0.20 mm <D_{60}<1.00 mm and "Cu" between 2 to 5) with corrected SPT (N) <=15 iii. Clean Sand granular soil with corrected SPT (N) value more than 30 i.e. $(N1)_{60CS}$ > 30 are non-liquefiable	tion which may cause excessive total and differential settlement due to vibration under earthquake. ii. Liquefaction susceptibility for the cohesive soil under earthquake is assessed as per the following criteria • Liquefiable if the LL<37%, PI (Ip)<12% and NMC>0.85 LL • Marginally Liquefiable if the LL<47%, PI<20% and NMC>0.85 LL • Non-Liquefiable if the LL>47%, PI>20% and NMC<0.85 LL

Hence based on the above stated "Indian" guidelines the liquefaction susceptibility of the foundation soil layers as explored from the specified boreholes along the said "ROB" structure are examined.

From the "Subsurface Stratification" as specified under "Table 1" before, it can be found that the layers of loose to medium dense non-plastic silty sand (SM) and / poorly graded sand (SP) having low value of field recorded SPT (N) (say N<30) are present at top up to 7.00 to 10.00 from the existing ground level in most of the boreholes at the substructure locations along the ROB. The ground water table was also found quite close (within 2.15 m to 4.65 m from EGL) to the top of boreholes at the time of exploration work which may often rise up to the existing ground level (EGL) during monsoon and may remain at that raised level even after the monsoon for some period in every year. The natural moisture content (NMC) of the explored top "non-plastic NP loose to medium dense sandy soil layer" was found quite high and close to the liquid limit (LL) which may get completely saturated during and immediately afterthe monsoon period. Since the project road is in "Seismic Zone IV" having moderate to high seismicity, so the existing top "loose to medium denes non-plastic (NP) silty sand (SM) / poorly graded sand (SM) layers" at the foundation locations of the said

ROB structure may lose significant part of its shear strength and get liquefied under saturation due to any probable occurrence of earthquake.

So the evaluation of "Liquefaction Potential" and the assessment of post occur- rence effect of "Liquefaction" under seismic condition for the explored foundationsoil layers at the substructure locations of ROB were very much important and neces- sary for ensuring the long-term stability and safety of the shallow foundation by adopting the suitable anti-liquefaction measures of the existing ground.

3.3 Evaluation of Liquefaction Potential of Soil

The evaluation of "Liquefaction Potential" was carried out following the procedure given in "Indian Guidelines" namely (i) "Annex F" under "Clause No. 6.3.5.3" of "IS 1893 (Part 1):2016 (Reaffirmed 2021)" [1], (ii) "Appendix - A5" under "Clause No. 8.4.4" of "IRC:SP:114-2018" [2] and (iii) "Clause No. 3.9 of "IRC:75-2015" [3]. The evaluation process prescribed in all the "Indian Guidelines" said before are mostly based on the "Simplified Procedure" methodology developed by well renowned ex- pert Professors H.B. Seed and I.M. Idriss (1982) [12] [13] and its progressive revised, extended and refined version made by various researchers (Seed et al. 1983, 1985, Seed and De Alba 1986, Liao and Whitman 1986, Marcuson 1990, Seed and Harder 1990, Youd and Idriss 1997, Youd T.L. 2001 [14] , Idriss I. M. and Boulanger R.W. 2004 [9] and others). This simplified procedure of liquefaction potential assessment isbased on the one of most popular field penetration test namely "SPT (Standard Pene- tration Test (N)" records and it has become "Standard of Practice (SOP)" all over the world as it was declared in the "1996 Workshop" sponsored by "National Center for Earthquake Engineering Research (NCEER)" (Youd and Idriss 1997) [10] [11].

The standard procedure of evaluation of "Liquefaction Potential" involves determina- tion of following two important variables namely

1) "Earthquake Induced Cyclic Shear Stress" within the soil layers expressed as "Cy- clic Stress Ratio (CSR)" and
2) "Capacity of the soil layers to resist liquefaction" expressed as "Cyclic Resistance Ratio (CRR)".

The liquefaction of soil layers will only occur if the "Cyclic Shear Stress (CSR)" in- duced by earthquake is more than the mobilized "Cyclic Shear Resistance (CRR)". So the minimum required "Factor of Safety (FOS)", which is defined by "(CRR/CSR)" against any occurrence of liquefaction, was considered as 1.00 [1].

The "Cyclic Stress Ratio (CSR)" and "Cyclic Resistance Ratio (CRR)" of any soil layer and the corresponding "Factor of Safety (FOS)" against any occurrence of "Liq- uefaction" for the soil layer can be evaluated by following the detailed stepwise pro- cedure given in any of the Indian guidelines referred before.

Here, the estimation procedure of "Liquefaction Potential" as specified in the BIS guidelines namely "IS 1893 (Part 1): 2016 (Reaffirmed 2021) [1]" under its "Annex F" of "Clause No. 6.3.5.3" is briefly described below.

The "Cyclic Stress Ratio (CSR)" induced by the earthquake is evaluated using the following expression as given in the "BIS Guideline".

- **Cyclic Stress Ratio i.e. CSR = $[\tau_{av} / \sigma'_{v0}] = [0.65 \times (a_{max}/g) \times (\sigma_{v0} / \sigma'_{v0}) \times r_d]$,** where, (a_{max}/g) = Peak Ground Acceleration (PGA) to Gravity i.e. in terms of "g",

$(\sigma_{v0} / \sigma'_{v0})$ = [Total Overburden Pressure i.e. σ_{vo} / Effective Overburden Pressure i.e. σ'_{vo}]

r_d = Stress Reduction Co-efficient (Accounts for Flexibility of Soil)
= [1.00 - 0.00765z] for z < 9.15 m,
= [1.174 - 0.0267z] for 9.15 m < z < 23 m,
= [0.744 - 0.008z] for 23 m < z < 30 m,
= 0.50 for z > 30 m, where z is the depth below ground surface

Similarly, the "Cyclic Resistance Ratio (CRR)" corresponding to the shear resistance mobilized for the considered soil layer is evaluated using the following expressions as per the "BIS Guideline".

- **Cyclic Resistance Ratio i.e. CRR for Earthquake Magnitude (M_w) = 7.5 i.e.**

$CRR_{7.5} = 1/ [34-(N_1)_{60CS}] + [(N_1)_{60CS} / 135] + 50/ [10 \times (N_1)_{60CS} +45]^2 - (1/200)$

where, $(N_1)_{60CS}$ = SPT value normalized to and "Equivalent Clean Sand" Value

i.e. $(N_1)_{60CS} = [\ \alpha + \beta*(N_1)_{60}\ ]$,

$\alpha = 0$ & $\beta = 1.0$ for Fine Content (FC) <= 5%,
$\alpha = e^{[1.76 - (190/FC^{\wedge}2)]}$ & $\beta = [0.99 + FC^{1.5}/1000]$ for Fine Content 5% < (FC) < 35%,
$\alpha = 5.0$ and $\beta = 1.2$ for Fine Content (FC) >= 35%
$(N_1)_{60}$ = Corrected SPT corresponding to 60% of Energy with correction due to "Overburden Pressure" = $[C_N \times N_{60}] = [C_N \times (C_{60} \times N)] = [C_N \times (C_{HT} \times C_{HW} \times C_{SS} \times C_{RL} \times C_{BD}) \times N]$

- For cohesionless (sandy) soil i.e. other than silty/clayey soil namely other than CL, CI, CH, ML, MI, MH, corrections to measured/uncorrected i.e. "N" to reference "Effective Overburden Pressure" σ'_{v0} of approximately 100 kPa i.e. 10 t/m^2, i.e. $C_N = (10/\sigma'_{v0})^{0.50}$ & $C_N <= 1.70$
- For Standard SPT as per IS 2131, C_{60} = 1.00, otherwise $C_{60} = (C_{HT} \times C_{HW} \times C_{SS} \times C_{RL} \times C_{BD})$
- C_{HT} : For "Donut Hammer with Rope and Pulley" = 0.75 and for "Donut Hammer with trip / Auto" = 1.33
- C_{HW} = [H*W / 48387], where H is the "Height of Free Fall of Hammer" in "mm" and W is the "Weight of Hammer" in "Kg"
- C_{SS} = 1.10 and 1.20 respectively for "Loose" and "Dense" Sand in case of Standard Samples with room for Liners, but used without Liners
- C_{SS} = 0.90 and 0.80 respectively for "Loose" and "Dense" Sand in case of Standard Samples with room for Liner and Liners are used
- C_{RL} = 0.75 for 0.0 m to 3.0 m, 0.80 for 3.0 m to 4.0 m, 0.85 for 4.0 m to 6.0 m, 0.95 for 6.0 m to 10.0 m and 1.00 for 10.0 m to 30.0 m
- C_{BD} = 1.00 for 65 - 115 mm, 1.05 for 150 mm and 1.15 for 200 mm diameter

Then, the "Cyclic Resistance Ratio (CRR)" for specific earthquake magnitude (M) is estimated using the following expression given in the referred "BIS Guideline".

- **Cyclic Resistance Ratio (CRR) i.e. CRR = $[K_\sigma \times K_\alpha \times MSF \times CRR_{7.5}]$**

where, K_σ = Correction for "High Overburden Stresses" for depth more than 15 m,

= $[\sigma'_{v0} / P_a\ (= 10\ t/m^2)]^{(f-1)}$, where "f" is an exponent & its value depends on "Relative Density (Dr)", For "D_r" = 40% to 60%, f = 0.80 ~ 0.70 and for "D_r" = 60% to 80%, f = 0.70 ~ 0.60

K_α = Correction for "Static Shear Stresses" required only in case of "Sloping Ground"

= 1.00 (For Routine Engineering Practice for other than sloping ground

MSF = Earthquake Magnitude Scaling Factor = $[10^{2.24} / M_W^{2.56}]$

Finally, the "Factor of Safety i.e. FOS" against any "Liquefaction" is estimated as the ratio of "Cyclic Resistance Ratio (CRR)" and "Cyclic Stress Ratio (CSR)" i.e. FOS = [CRR / CSR]. As mentioned earlier, the minimum required factor safety (FOS) against the probable occurrence of "Liquefaction" was satisfied as "1.00" [1].

As per the above stated procedure given in the relevant "BIS Guideline", the analysis of "Liquefaction Potential" was done for the foundation soil strata stated earlier in "Table 1" under "Subsurface Stratification" at the substructure locations of the ROB structure. For doing the analysis of "Liquefaction Potential" of the ground along the ROB structure, the position of "Ground Water Table (GWT)" was considered at the existing ground level corresponding to the critical condition during and immediately after monsoon. The analysis of "Liquefaction Potential" for the foundation soil at substructure locations of ROB structure was carried out considering the "Seismic Zone IV" and the "Peak Ground Acceleration" as "0.24g" corresponding to the "Maximum Considered Earthquake (MCE)" having magnitude of earthquake in "Richter Scale" i.e. M_W as "6.50" following the guidelines of "BIS" mentioned earlier. It is to be noted that the analysis of "Liquefaction Potential" of the ground along the pro- posed "ROB Structure" was done for the condition of "In-situ Free Ground".

The "Factor of Safety (FOS)" against any probable occurrence "Liquefaction" and the corresponding liquefiable depth the boreholes at the respective substructure (piers and abutments) locations long the "ROB 1" structure is summarised in **Table 3** below.

Table 3. Results of Liquefaction Potential Evaluation for Subsurface Layers at "ROB 1"

Substructure No.	Borehole No.	Depth of Liquefiable Soil (m)	Type of Liquefiable Soil (as per IS)	Range of Factor of Safety (FOS) against "Liquefaction"
Pier, P10	BH-ROB-1/8	7.00 (1.50 m thick Non-Liquefiable layer from 2.50 to 4.00 m)	SM/SP	0.56 - 0.93 (From 2.50 to 4.00 m, FOS 1.19)
Pier, P14	BH-ROB-1/12	7.00	SM/SP	0.29 - 0.75
Pier, P15 & P16	BH-ROB-1/14	8.50	SP	0.24 - 0.83
Pier, P18 & P19	BH-ROB-1/17	7.00	SP	0.24 - 0.90
Pier, P20	BH-ROB-1/18	7.00	SP	0.34 - 0.90
Pier, P21	BH-ROB-1/19	10.00 (1.50 m thick Non-Liquefiable layer from 5.50 to 7.00 m)	SP	0.21 - 0.90 (From 5.50 to 7.00 m, FOS 1.34 - >1.00)

Substructure No.	Borehole No.	Depth of Lique-fiable Soil (m)	Type of Lique-fiable Soil (as per IS)	Range of Factor of Safety (FOS) against "Liquefaction"
Pier, P23	BH-ROB-1/21	7.00 (1.50 m thick Non-Liquefiable layer from 4.00 to 5.50 m)	SP	0.46 - 0.99 (From 4.00 to 5.50 m, FOS 1.23)
Pier, P24	BH-ROB-1/22	7.00	SP	0.29 - 0.99
Pier, P25	BH-ROB-1/23	10.00 (3.00 m thick Non-Liquefiable layer from 4.00 to 7.00 m)	SP	0.29 - 0.74 (From 4.00 to 7.00 m, FOS >1.00)
Abutment, A2	BH-ROB-1/24	10.00 (1.50 m thick Non-Liquefiable layer from 7.00 to 8.50 m)	SP	0.34 - 0.99 (From 7.00 to 8.50 m, FOS >1.63)
	BH-ROB-1/25	7.00	SP	0.29 - 0.98

From the above **Table 3** it can be found that the ground at the substructure (piers and abutment) locations along the "ROB 1" was mostly liquefiable for minimum 7.00 m to maximum 10.00 m of depth from the exiting ground level. The corresponding "Factor of Safety (FOS)" against the occurrence of "Liquefaction" was having the range 0.21 - 0.93 which was evaluated through the analysis of "Liquefaction Poten- tial" for the "In-situ" condition of ground using the foundation soil layer profiles of the available boreholes along the "ROB 1" structure of the project. So the suitable ground treatment measure in appropriate engineering manner was required to beadopted to make the ground at the foundation locations of the substructure "non- liquefiable" under seismic condition. Again, the anticipated deformation of the in-situ ground mostly in the form of "vertical settlement" at the foundation locations of the substructures along the "ROB 1" structure was essentially required to be estimateddue to any probable occurrence of liquefaction under seismic condition.

3.4 Evaluation of Anticipated Liquefaction Induced Ground Deformation

Since the ground for the project area was almost open at its earlier condition and this open free ground was prepared for placing the substructure along the alignment of ROB, so the ground deformation due to the probable occurrence of liquefaction wouldbe mostly in the form ground settlement in post-earthquake condition.

Post-Earthquake Settlement of Liquefied Soil. The post-earthquake densification i.e. liquefaction induced settlement of any cohesionless soil like sand is controlled by the initial state say density of the sand, the maximum shear strain induced in the sand in addition to the amount of excess pore pressure generated by the earthquake. It is found from the laboratory experiments that the volumetric strain after initial liquefaction varies with relative density and maximum shear strain of the soil. The post liquefaction settlement of any plain ground sites can be determined by using the "Design Chart" as developed by "Tokimatsu" and "Seed" (1987) [5] [6] based on the experimental studies. The design chart proposed by "Tokimatsu" and "Seed" (1987) [5] [6] is for the earthquake magnitude (M_w) of 7.50 as shown in "Figure 1".

For earthquakes of any other magnitudes, an equivalent "Cyclic Stress Ratio (CSR_M)", is required to be determined using the applicable relevant equation. For estimating the post-earthquake i.e. liquefaction induced settlement of any foundation soil deposits, the volumetric strains corresponding to the corrected SPT blow counts i.e. "$(N_1)_{60}$" and "Cyclic Stress Ratios (CSR)" for each of the liquefied soil layers in the deposits is to be multiplied by the corresponding layer thickness. For the soil lay- ers with fines, the "SPT value" normalized to the "Equivalent Clean Sand" value i.e. "$(N_1)_{60CS}$" in place of "$(N_1)_{60}$" is used. It is to be noted that these estimation of settlement is valid for levelled i.e. plain ground sites which have no potential for lateral spreading due to the earthquake. The estimated settlement is likely to be larger for lateral spreading sites than those for plain ground.

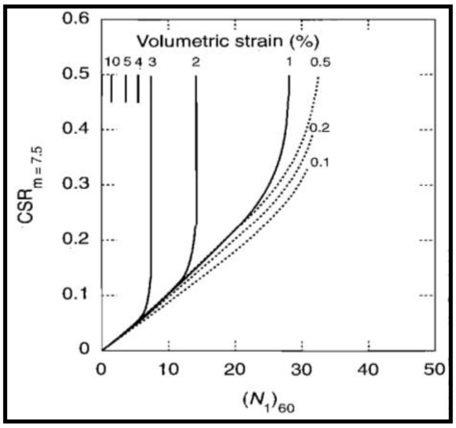

Figure 1: Chart for estimation of volumetric strain in saturated sand from cyclic stress ratio and standard penetration resistance (After Tokimatsu and Seed, 1987, Estimation of settlements in sand due to earthquake shaking, JGE, Vol 113, No. 8) [5] [6]

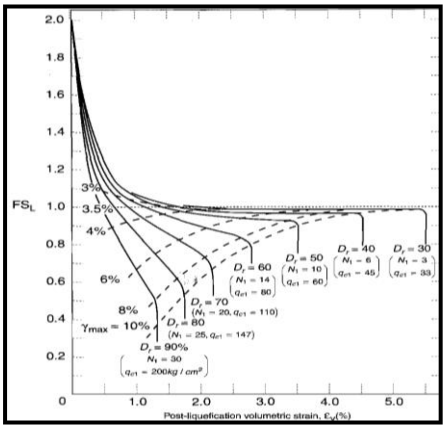

Figure 2: Chart for estimating post-liquefaction volumetric strain of clean sand as function of factor of safety against liquefaction or maximum shear strain (After Ishihara and Yoshimine, 1992, JJSMFE) [5] [6]

Alternatively, the liquefaction induced settlement of any foundation soil layer can be estimated through determination of the post liquefaction volumetric strain by using either the factor of safety against liquefaction or the maximum cyclic shear strain, and the relative density, SPT resistance, or CPT tip resistance as it is proposed by "Ishihara" and "Yoshimine" (1992) [5] [6] shown in the "Figure 2". Here, the "$(N_1)_{60}$" values is to be converted to "N_1" values [Since the "N_1" represents "Japanese SPT" which typically transmit 20% more energy to the conventional standardized SPT sampler; hence "N_1" = 0.833*$(N_1)_{60}$]. For using the chart in case of soil layers with fines, the "SPT value" normalized to the "Equivalent Clean Sand" Value i.e. "$(N1)_{CS}$"in place of "N_1" is used. Finally, the liquefaction induced ground surface settlement is obtained by integrating of the volumetric strain which is obtained through the "Design

Chart" of "Ishihara" and "Yoshimine" (1992) [5] [6] over thickness of the liquefied depth of layers.

Based on the above stated two methods, the liquefaction induced settlement of the ground surface for all the boreholes along the substructure locations of the "ROB 1" structure was estimated. The estimated anticipated settlement of the free ground surface in original condition for each of the boreholes along the "ROB 1" structure is summarised in **Table 4** below.

Table 4. Results of Anticipated Liquefaction Induced Ground Deformation for Subsurface Layers at "ROB 1"

Substructure No.	Borehole No.	Depth of Liquefiable Soil (m)	Type of Liquefiable Soil (as per IS)	Anticipated Liquefaction induced Settlement of Ground Surface (mm)		
				Method A: "Tokimatsu and Seed Approach (1987)"	Method B: "Ishihara and Yoshimine Approach (1992)"	Average value
Pier, P10	BH-ROB-1/8	7.00	SM/SP	78.75	94.00	86.38
Pier, P14	BH-ROB-1/12	7.00	SM/SP	139.50	215.75	177.63
Pier, P15 & P16	BH-ROB-1/14	8.50	SP	170.75	253.00	211.88
Pier, P18 & P19	BH-ROB-1/17	7.00	SP	147.75	199.75	173.75
Pier, P20	BH-ROB-1/18	7.00	SP	116.50	189.25	152.88
Pier, P21	BH-ROB-1/19	10.00	SP	148.25	144.00	146.13
Pier, P23	BH-ROB-1/21	7.00	SP	83.00	110.00	96.50
Pier, P24	BH-ROB-1/22	7.00	SP	104.25	155.00	129.63
Pier, P25	BH-ROB-1/23	10.00	SP	93.00	136.25	114.63
Abutment, A2	BH-ROB-1/24	10.00	SP	110.25	131.00	120.63
	BH-ROB-1/25	7.00	SP	105.75	146.25	126

From the above **Table 4** above, it is noted that the value of anticipated liquefaction induced settlement of the ground surface along the "ROB 1" structure as per the method proposed by "Tokimatsu and Seed (1987)" [5] [6] was estimated as varying from 73.75 mm to 253 mm. Similarly, the estimated value of post-earthquake settle- ment for the ground surface along the "ROB 1" structure as per the method proposed by "Ishihara and Yoshimine Approach (1992)" [5] [6] was found ranging from 63.25 mm to 211.88 mm. So the average value of anticipated liquefaction induced settlement of the ground surface at the substructure locations along the "ROB 1" was found within the range 68.50 mm to 232.44 mm exceeding the permissible value of total settlement.

4. Ground Treatment For Liquefaction Mitigations of Soils

Since the in-situ condition of the existing soil which is mostly of non-plastic (NP) loose to medium dense cohesionless soil namely silty sand (SM) and/or poorly graded sand (SP) having ground water table close to the exiting borehole top found liquefia- ble maximum up to 10.00 m depth from EGL under saturation in seismic condition, so it was essentially required to do the necessary ground treatment of the foundation soil layers till up to the anticipated depth of probable liquefaction for placing the shallow i.e. open foundation instead of adopting the deep foundation namely pile foundation bypassing the underlying weak liquefiable layer and terminating the bottom / tip of the pile foundation into the non-liquefiable dense or hard strata after adequately penetrating into the same strata.

The main objective of commonly used methods for mitigation of seismic hazards is to reduce the tendency of the soil to generate positive excess pore water pressure during earthquake shaking as well as to increase the strength and stiffness of the foundation soil. To control or prevent the liquefaction susceptibility of foundation soil and restrict the liquefaction induced ground settlements to the acceptable serviceable limit, the suitable site remediation measures involving one or more ground treatment methods were required to be adopted. The available various ground treatment methods of controlling liquefaction have a very wide variation in costing. Based on the mechanism of ground treatment works for mitigating the liquefaction hazards under seismic condition, the commonly available methods are usually divided into following four major categories.

a. Densification Methods (For shallow depth either by loosening & recompaction or by replacement, vibro-compaction, dynamic compaction, blasting)
b. Reinforcement Methods (Stone Columns, Compaction Piles, Drilled Inclusion like Micropile etc.)
c. Grouting / Mixing Methods (Different Methods of Grouting, Soil Mixing, Jet Grouting, Compaction Grouting etc.)
d. Drainage Methods (Stone Column, Gravel Drain)

Among all the above stated different methods of ground treatment, the "Densification Method" was preferred from the consideration of effectiveness of the method to the existing soil in addition to the requirements of lesser amount of time and overall cost.

4.1 Densification Method of Ground Treatment

Densification is one of the best and regularly utilized methods of improving soil engineering nature for reducing or nullifying the seismic hazards. Simultaneously, it is to be noted that the increased stiffness of a densified soil mass will make it react contrastingly to seismic tremor movement; displacement amplitudes are probably going to diminish, however accelerations may be fairly more than they would have been had the soil mass not been improved. Densification results into permanent volume changes which produce settlement of the ground surface.

Densification by Vibro-compaction or Vibro-Probing. The vibro compaction process subjects the soil mass to high accelerations during compaction. These levels of dynamic strain are unlikely to be repeated, even under earthquake loading. In vibro compaction, when vibrations are passed into the soil, the compaction occurs through the rearrangement of soil particles. The vibro compaction is possible mostly in cohesionless soils with less than 20% fines (say finer than 75 micron size). The vibro compaction is usually not found suitable in cohesive, fine-grained soils. In cohesive soils, the cohesion between the particles prevents rearrangement and compaction from occurring. Therefore, the vibro compaction is found applicable for loose to medium dense cohesionless soil [15] [16]. The "Figure 3" [8] shows the zone of fine grained sandy soil with some silt content which is susceptible to liquefaction and applicable method of ground treatment.

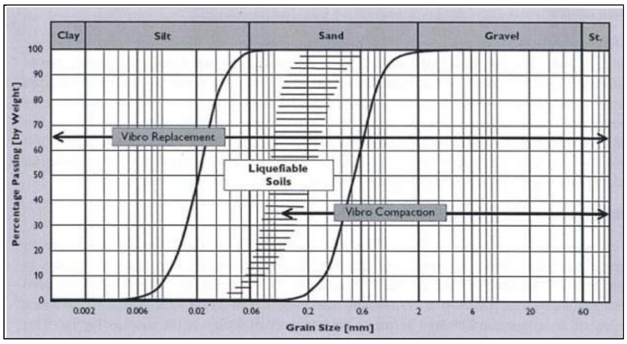

Figure 3: Liquefiable Soil and applicable ranges of Deep Vibratory Compaction Techniques (Keller, 2012) [8]

Since the foundation soil at the substructure locations along the "ROB 1" is either "Silty Sand (SM)" or "Poorly Graded Sand (SP)" with occasional presence of the other intermediate layers of "Sandy Silt (ML/MI)", so the "Densification" by "Vibro-compaction Method" were suitably adopted for the mitigation measures against any probable occurrence of liquefaction of ground at the said structure in the project.

Vibro-compaction by Driving and Withdrawing Section of Sheet Pile. The section of steel sheet pile which is generally used for retaining the sides of open excavation was driven by inducing the heavy vibration at the top by using suitable vibrating hammer. The process of driving the section of sheet pile up to the desired depth of insertion at initial stage and then withdrawing the same sheet pile section gradually from the ground by applying the required level of vibration using vibro-hammer had quite similar effect of vibro-compaction which helped for the densification of soil till up to the depth of sheet pile installation. The process of vibro-compaction was used to in- crease the relative density of the non-plastic saturated loose to medium dense cohe- sionless sandy soil. In vibro-compaction the horizontal vibrations are induced in asso- ciation with fluid to reduce the inter-particle friction of the surround soil. This helps the material to densify and forms a column of soil with improved engineering proper- ties including an increase in strength and reduction in compressibility. The principle of sand compaction (in vibro-flotation) [15] [16] consists of flotation of soil particleas result of vibration when then allows for rearrangement of particle in to a denser state (Moretrench, 2012). As the ground water table was found very close to the exist-ing ground level along the "ROB 1" structure location, so the site was mostly saturated up to original ground level. To densify the exiting loose to medium dense saturated cohesionless soil maximum up to the depth of 12.00 m below the top ground surface, the adequate level of vibration was induced through installation of the section of sheet pile up to the desired depth. Since the depth of maximum 10 m wider open foundation for the substructures of the “ROB 1” structure were kept 4 m below the original ground level, so the soil of minimum 0.5 times of width foundation say 5 m below thefounding level was targeted to be treated with adopted vibro-probing method of densi-fication. The section of steel sheet pile was driven at first and then gradually with- drawn through the marked points in triangular pattern with center to center spacing of 2 m covering the entire foundation area and some portion beyond the demarcated foundation base area for the locations of substructures (pier and abutment) along the “ROB 1” in the project. The vibro-hammer of 4 MT of capacity along with 50 MT of capacity crane for lifting and installing the section of steel sheet pile were used in the project. The saturated ground created a “Quick Sand” (temporarily liquefying the material) condition which allowed the sheet pile section to sink inside the ground under vibration. The vibration was also induced after penetrating desired depth of 12 m for about 5 minutes of duration for achieving better densification of the entire treat-ed depth of foundation soil. The driven sheet pile section was withdrawn back in lifts of 0.50 m interval to enhance the compaction of surrounding soil. During withdrawal of sheet pile section in short lifts and re-penetration of the same was repeated to den- sify the foundation soil. The rate of withdrawal from the ground and re-penetrationsof sheet pile sections into the ground were maintained to achieve uniform densifica- tion of the soil. The following “Photograph 1” and “Photograph 2” show the adopted method of densification namely “vibro-compaction” or “vibro-probing” by driving and withdrawing of “Section of Steel Sheet Pile” with the help of “vibro-hammer” placed over the steel sheet pile.

Photograph 1: Ground Treatment by Vibro-probing with Section of Steel Sheet Pile at the marked grid points covering Foundation Base and its beyond area (Courtesy: M/s. L&T Construction, Transportation Infrastructure)

Photograph 2: The driving and withdrawal of Section of Steel Sheet Pile with Vibro-hammer by placing it at top of Sheet Pile (Courtesy: M/s. L&T Construction, Trans-portation Infrastructure)

The subsidence of the ground surface due to the vibro-compaction effect by driving and withdrawing the section of steel sheet pile was filled with good earth at ground level and levelled before placing the actual RCC foundation slab for the substructure locations of the “ROB 1” structure. The materials used for backfilling of the craters created due to the densification of existing loose to medium dense NP cohesionless soil using vibro-probing by section of steel sheet piling was natural ground material sufficiently hard, chemically inert and appropriately graded.

Quality Control Measures of Ground Treatment Work of Vibro-compaction. The quality control check of the adopted densification method of vibro-compaction was very much necessary. It was important to ensure that the method of vibro-probing as used was operated efficiently and effectively for densifying the loose to medium dense portion of the deposits which was vulnerable to liquefaction under seismic condition. At the end of vibro-probing method of densification through the process of driving and withdrawing the section of steel sheet pile in subsequent controlled manner, the densities of existing soil were normally checked to ensure that the adequate compaction of the top minimum 10 to 12 m of ground was achieved. The improvement in the densities of the existing top soil till up to the maximum desired depth of 12 m was checked by conducting the field test namely SPTand recording the related SPT (N) val- ues. The typical photo of field soundingtest namely "Standard Penetration Test (SPT)" in progress for the foundation plan area at the substructure locations of "ROB 1" structure is shown in "Photograph 3".

Photograph 3: Showing Field SPT in progress at the foundation base area of substructure after "Vibro-probing" method of densification (Courtesy: M/s. L&T Construction, Transportation Infrastructure)

The comparison of the field recorded SPT (N) values in between "Original" and "Treated" ground conditions for top subsurface layers at the locations of substructures along "ROB 1" structure is given in the **Table 5** below.

Table 5. Comparison of Field Recorded SPT (N) Values in between Original and Treated Conditions for Top Subsurface Layers at "ROB 1"

Substructure No.	Depth of Liquefiable Soil in Original Condition without any Ground Treatment (m)	Type of Liquefiable Soil (as per IS)	Details of Recorded SPT (N) within Liquefiable Zone in Original Condition without any Ground Treatment		Details of Recorded SPT (N) within Liquefiable Zone after Ground Treatment by Densification using Vibro-probing	
			Distribution of SPT (N) Values	Average Value of Recorded SPT (N) Values	Distribution of SPT (N) Values	Average Value of Recorded SPT (N) Values
Pier, P10	7.00	SM/SP	7/10/10/12	~10	15/17/18/20	~18
Pier, P14	7.00	SM/SP	1/3/4/11	~5	Pile Foundation	
Pier, P15 & P16	8.50	SP	2/3/9/12/12	~8	15/18/19/20/20	~19
Pier, P18 &	7.00	SP	2/3/9/13	~7	16/18/18/20	~18

Substructure No.	Depth of Liquefiable Soil in Original Condition without any Ground Treatment (m)	Type of Liquefia-ble Soil (as per IS)	Details of Recorded SPT (N) within Liquefiable Zone in Original Condition without any Ground Treatment		Details of Recorded SPT (N) within Liquefiable Zone after Ground Treatment by Densifica-tion using Vibro-probing	
			Distribution of SPT (N) Values	Average Value of Recorded SPT (N) Values	Distribution of SPT (N) Values	Average Value of Recorded SPT (N) Values
P19						
Pier, P20	7.00	SP	4/5/8/13	~8	15/18/19/20	~18
Pier, P21	10.00	SP	1/12/12/17/25/14	~13	16/17/18/20/30/28	~22
Pier, P23	7.00	SP	6/11/15/14	~12	16/18/18/19	~18
Pier, P24	7.00	SP	3/9/11/14	~9	17/19/21/25	~21
Pier, P25	10.00	SP	3/10/22/23/26/13	~16	15/18/30/30/30/25	~25
Abutment, A2	10.00	SP	4/13/13/14/21/17	~14	16/20/20/21/30/27	~22
	7.00	SP	3/13/12/9	~9	16/25/27/25	~23

From the above table it can be seen that there are significant increases of the field recorded SPT (N) values for the top subsurface layers in each and every substructure locations along the “ROB 1” structure due to the “Vibro-Probing Method” of densification by driving and withdrawing of the section of steel sheet pile covering foundation base area. From the above table it is to be noted that the range of average values of field SPT (N) was recorded in between 5 to 16 in various boreholes in original ground condition whereas the average field recorded SPT (N) values was found in- creased varying from 18 to 25 in post ground densification condition.

4.2 Assessment of Anticipated Liquefaction Potential and its Post Occurrence Effect for Treated Foundation Soil

The anticipated “Liquefaction Potential” and the post-occurrence effect of “Liquefaction” for the foundation soil at the substructure locations of the “ROB 1” structure were assessed based on the field recorded SPT (N) values for the top subsurface lay- ers as obtained after doing the “Densification Method” of ground treatment by “Vi- bro-Probing” i.e. "Vibro-compaction” described before. The simplified procedure of “Evaluation of Liquefaction Potential” as stated earlier under “Article 3.3” following the relevant “BIS guideline” namely “IS 1893 (Part 1): 2016 (Reaffirmed 2021)” [1] and the procedure of “Evaluation of Liquefaction Induced Ground Deformation” as described under “Article 3.4” following the methods proposed by “Tokimatsu and Seed (1987)” [5] [6] and “Ishihara and Yoshimine (1992)” [5] [6] were used for esti- mating seismic hazards of the treated foundation soil for all the substructure locations along the “ROB 1”. The summary of estimations of the anticipated “Liquefaction

Potentials" and "Liquefaction Induced Settlement" for the treated ground at each of the substructure locations along "ROB 1" structure is summarised in **Table 6** below.

Table 6. Summary of Anticipated "Liquefaction Potential" and "Liquefaction Induced Settlement" in Treated Conditions for Subsurface Layers at "ROB 1"

Substructure No.	Type of Originally Liquefiable Soil (as per IS)	Range of Factor of Safety (FOS) against "Liquefaction"	Depth of Liquefiable Soil Layer (if any)	Liquefaction induced Settlement of Ground Surface (mm)		
				Method A: "Tokimatsu and Seed Approach (1987)"	Method B: "Ishihara and Yoshimine Approach (1992)"	Average value
Pier, P10	SM/SP	1.39 - >1.00	0.00	12.5	4.00	8.25
Pier, P14	SM/SP	Pile Foundation of adequate length instead of Open Foundation				
Pier, P15 & P16	SP	1.29 - >1.00	0.00	20.25	11.50	15.875
Pier, P18 & P19	SP	1.42 - >1.00	0.00	11.75	16.25	14.00
Pier, P20	SP	1.25 - >1.00	0.00	15.00	6.88	10.94
Pier, P21	SP	1.37 - >1.00	0.00	13.80	7.75	10.775
Pier, P23	SP	1.42 - > 1.00	0.00	12.00	5.50	8.75
Pier, P24	SP	1.74 - >1.00	0.00	11.50	2.00	6.75
Pier, P25	SP	1.25 - >1.00	0.00	14.00	7.25	10.625
Abutment, A2	SP	1.42 - > 1.00	0.00	12.00	2.50	7.25
	SP	1.42 - >1.00	0.00	12.00	2.50	7.25

From the above table it can be seen that the subsurface layers at the substructure locations of said "ROB 1" were modified to completely non-liquefiable after doing the ground treatment by "Vibro-Probing Method" of "Densification" and there are adequate (>1.00) "Factor of Safety (FOS)" against any probable occurrence of "Liquefaction" under "Seismic Condition". The average value of anticipated vertical deformation i.e. settlement of the top non-liquefiable soil layers due to the probable level of "Cyclic Straining" during seismic condition at the substructure locations along the "ROB 1" structure were found within the range 6.75 mm to 15.875 mm which was within the maximum permissible limit for laying the "RCC Open Foundation". So, the stabilities of the "RCC Open Foundations" for all the substructures other than the locations carrying the span over the railway tracks along the said "ROB 1" structure were well ensured under seismic condition in addition to the stabilities of open foundations in static conditions by adopting the "Liquefaction Mitigation Measures" namely "Vibro-compaction Method" of "Ground Densification" as stated in the paper.

5. Summary

This paper states about the case study on assessment of liquefaction potential of loose to medium dense cohesionless sandy foundation soil deposits having high water table at one ROB structure namely "ROB 1" along the project road of NH-31D from

"Siliguri (Ghoshpukur)" to "Alipurduar (Salsalabari)" in the northern part of "West Bengal, India". Since the said project corridor is situated in the "Seismic Zone IV" as per "IS 1893 (Part 1): 2016 (Reaffirmed 2021)" [1], so there was high susceptibility of liquefaction of the ground composed of loose to medium dense saturated cohesion-less sandy soil under the open foundations of some substructures along the "ROB 1" structure. The open foundations were not feasible due to the presence of considerable liquefiable depth of soil beyond the proposed founding level at those identified locations of substructures along the "ROB 1". So it was necessary to do the appropriate mitigation measure against possible liquefaction of the foundation soil deposits below the substructures. The "vibro-probing" i.e. "vibro-compaction" method of densification for loose to medium dense saturated sandy soil was finally adopted as an effec- tive and economic mitigation measure against probable liquefaction in seismic condi- tion. The paper describes about the said "vibro-probing method" of densification for loose to medium dense saturated non-plastic (NP) sandy deposits under the open foundations of the substructures located along the said structure of "ROB 1". Thefield sounding by "Standard Penetration Test (SPT)" method was conducted at the substructure locations of "ROB 1" structure and the anticipated liquefaction potential including the post-occurrence effect for the foundation soil layers were also assessedat the end of ground treatment to verify the efficiency of the adopted mitigation measures against any probable occurrence of liquefaction during earthquake.

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