Studies on Liquefaction for Foundation Systems of Pellet Plant at Paradeep

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ABSTRACT: Many liquefaction induced foundation failures are reported during past earthquakes. Scarcity of codal provisions in India for quantifications of liquefaction potential for a site, where industrial foundations are being constructed, makes the problem challenging to design engineers. In this paper, detailed investigation has been carried out to assess the liquefaction susceptibility of a site at Paradeep, Orissa, India during earthquake to select suitable foundation system. Liquefaction susceptibility studies based on SPT and CPT data has been carried out using the simplified procedure. From the analysis using SPT values, it has been found that the upper layers of silty sand / fine sand are prone to liquefaction for the ground water table at ground level. Based on calculated liquefaction potential analysis, the 0.5m to 0.6m diameter pile foundations are suggested up to 26m below ground level to transfer the loads in non-liquefiable stratum after confirming its capacity from load test data.

1 INTRODUCTON

The performance of super-structures during earthquakes is strongly influenced by the performance of the sub-soil layers. Local soil conditions can influence structural performance in two primary ways — by influencing the ground motions that excite the structure and by imposing additional deformations on the structure through ground failure. The first widespread observations of damage attributed to liquefaction were made in the 1964 Niigata, Japan (Seed and Idriss, 1966), and 1964 Alaska earthquakes. In several past earthquakes, liquefaction has been attributed to be responsible for significant damage to buildings and foundations. Liquefaction has been studied extensively over the past years, and substantial advances have been made in understanding the development and effects of this phenomenon on sub-soil strata. In the present paper, an actual field problem is assessed with proper guidelines for pile foundation using liquefaction studies.

It has been proposed to set up 8 MTPA (Metric Ton Per Annum) integrated iron ore pellet plant at site area near Paradeep, Orissa, India. The site area is primarily Deltaic Alluvial sediments drained by Mahanadi river near its confluence with the Bay of Bengal. Quantitative assessment of the likelihood of “triggering” or initiation of liquefaction is the necessary first step for this project to decide the suitable foundation system. Preliminary investigation revealed the presence of the fine to medium sand layer along with shallow water table depth in the region of proposed pellet plant. There are two general types of approaches for liquefaction potential analysis: (1) use of laboratory testing of “undisturbed” samples, and (2) use of empirical relationships based on correlations of observed field behavior with various in-situ “index” tests. As laboratory testing of “undisturbed” samples for liquefaction analysis are complex and involving substantial cost, the use of “index” tests are used for the present study.

1.1 *Geotechnical Investigations*

Because of its vicinity to the sea, area is low lying with little to very little undulations at places, mainly controlled by the topographical features. From the subsurface drilling carried out in the area under study, no rock was encountered within the drilled depth (i.e. 54m). This indicates deposition of a large volume of quaternary to sub-quaternary alluvial sediments in the confluence area by the river Mahanadi.

2 SUB-SOIL PROFILE

The sub-soil investigation work for the proposed site has been investigated by drilling ten (10) bore holes upto a maximum depth of 53.60m and three (3) bore holes upto a maximum depth of 30.12m below the existing ground level at specified locations. Additionally, seven (7) vane shear tests and four (4) Static Cone Penetration Tests (*SCPT*) were also conducted at specified locations. The details of soil layers like layer numbers, description of layers and the thickness of each layer as encountered in the bore holes (area wise) are typically presented in Table 1. In general, these silty sands contain fines ranging from 4% to 15%.

Table 1. Typical soil layers at Paradeep site

Layer No.	Description	Thickness (m)
Fill	Yellowish brown silty sand / fine sand	1.5-2.5
I	Soft / firm brownish dark grey silty clay	3.5- 5.5
II	Loose / medium silty sand / fine sand	10.5 -13.5
III	Yellow silty sandy clay / silty clayey sand	2.0-7.0
IV	Yellowish brown silty sand / fine /medium sand	4.5- 7.0
V	Yellowish grey sandy silty clay with blue patches	5.5 – 7.5
VI	Yellowish brown / silty sand / fine / medium sand	7.5 – 12.0
VII	Very stiff / hard dark grey / grey silty clay	3.5 – 6.00
VIII	Medium dense yellowish brown / radish yellow	2.00- 3.00

Standard Penetration Tests (*SPT*) were conducted as per Indian Standard IS: 2131-1981. Static Cone Penetration Tests (*SCPT*) were conducted at specified locations as per Indian Standard IS:4968 (Part-III)-1976. The field test data and graphical representations of cone resistances, frictional resistances and friction ratio with depth were plotted and are shown in Fig. 1. As the site is in the vicinity of the Mahanadi river and in the close proximity of sea front, the design groundwater table has been considered at the ground surface.

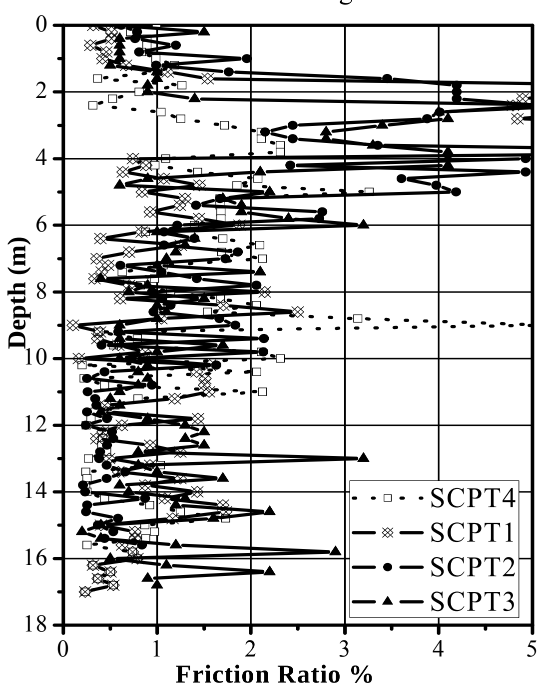

Fig. 1 Variation of depth vs. friction ratio (%) for different SCPT

3 ASSESMENT OF LIQUEFACTION SUSCEPTIBILITY

As per Indian Standard IS:1893 (Part-1):2002, liquefaction is likely in fine sands below water table with corrected SPT values less than 15 to about 5.0 m depth and less than or equal to 25 below 10.0 m depth (for Seismic Zone Levels III, IV and V). As per IS code, there is a potential for liquefaction of the soils to about 5 to 12 m depth. Typical observed and corrected N-value for some boreholes are plotted in Fig. 2. The particle size distribution curves of various samples (Fig. 3) tested also fell within the ranges of liquefiable soils specified by the Japanese Port Harbor Research Institute (JPHRI, 1989).

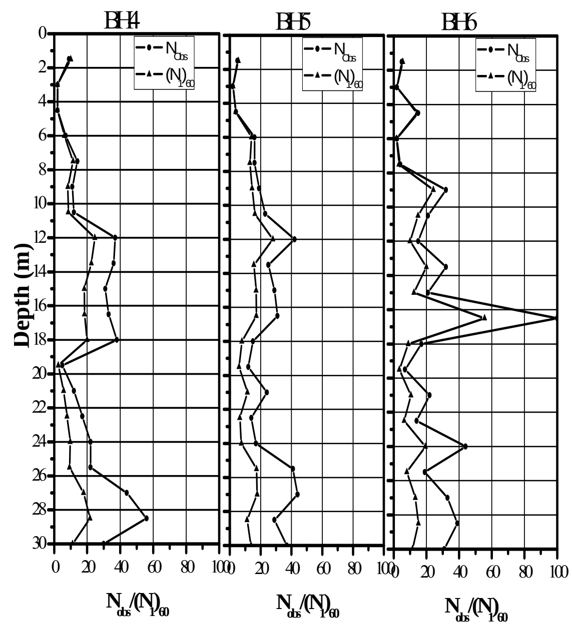

Fig. 2 Observed and corrected N-values at various depths for some typical boreholes

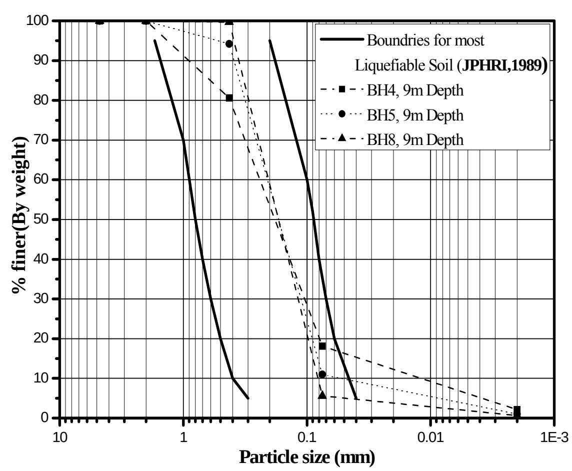

Fig. 3 Particle size distributions of samples for some typical boreholes

The most widely used method for evaluating liquefaction is the stress-based procedure first proposed by Seed and Idriss (1971). This empirical procedure was originally developed using observations of laboratory and field data, and has been continually refined by newer studies and by the increase in the number of liquefaction case histories (e.g., Seed et al., 1985; Ishihara, 1993; Youd and Idriss, 1997; Youd et al., 2001; Finn, 2002).

Detailed liquefaction analysis has been carried out based on the simplified procedure developed by Seed and Idriss (1971) and subsequent modifications as described by Youd et al. (2001).

As per the project specifications, the analysis has been carried out for design earthquake magnitude of 6.5 and peak ground acceleration of 0.16g for the Seismic Zone-III as per IS:1893 (Part-1)-2002. Estimation of two variables is required for evaluation of liquefaction resistance of soils: (1) the seismic demand on a soil layer (CSR) and (2) the capacity of the soil to resist liquefaction in terms of Cyclic Resistance Ratio ($CRR_{7.5}$).

The damage potential of earthquake ground motions is a function of both the amplitude and the duration of earthquake-induced motions (i.e., *demand*), is quantified in terms of cyclic stress ratio (*CSR*). In this analysis, the cyclic stress ratio (*CSR*) has been calculated for the selected peak horizontal ground accelerations at various depths using the following equations given in Seed and Idriss (1971).

$$CSR = \left(\frac{\tau_{av}}{\sigma'_{vo}}\right) = 0.65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_{vo}}{\sigma'_{vo}}\right) r_d \tag{1}$$

$$\begin{cases} r_d = 1.0 - 0.00765z & \text{for } z \le 9.15\text{m} \\ r_d = 1.174 - 0.267z & \text{for } 9.15\text{m} < z \le 23\text{m} \end{cases} \tag{2}$$

4 CYCLIC RESISTANCE RATIO (CRR)

In situ stress states for determination of $CRR_{7.5}$, field tests have become the state-of-practice for routine liquefaction investigation. *CRR* at the site has been computed based on SPT values and SCPT values based on the simplified procedure given by Youd et al. (2001). A Magnitude Scaling Factor (MSF) was applied to the $CRR_{7.5}$ values, to adjust the clean sand curves to the design earthquake magnitude of 6.5.

4.1 *CRR from SPT values*

As summarized in the recent state-of-the-art paper by Youd et al. 2001, in-situ test method like Standard Penetration Test (*SPT*) have now reached a level of sufficient maturity as to represent viable tools for estimation of $CRR_{7.5}$. An overburden stress correction (Kramer 1996) and correction for hammer energy ratio (ER) of 0.75 are also applied to normalize N_{60}. Other correction factors for borehole diameter, rod length, and samplers with or without liners have also been taken into account to finally calculate $(N_1)_{60}$. The SPT values - $(N_1)_{60}$ have further been corrected for fines content to get $(N_1)_{60cs.}$ The clean-sand base curve for determination of CRR based on corrected $(N_1)_{60}$, has been estimated by the following equation (Rauch, 1997) for $(N_1)_{60} \le 30$. For $(N_1)_{60} \ge 30$, clean granular soils are too dense to liquefy and are classed as non-liquefiable.

$$CRR_{7.5} = \frac{1}{34-(N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{\left[10.(N_1)_{60}+45\right]^2} - \frac{1}{200} \tag{3}$$

Plots of CSR and $CRR_{7.5}$ (based on SPT values) versus depth are presented in Fig. 4 for some typical boreholes.

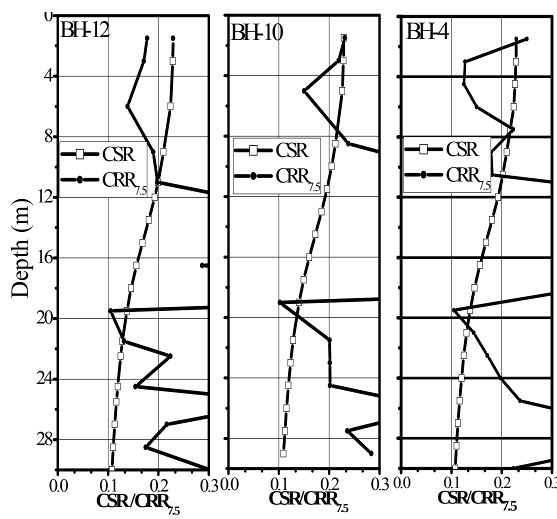

Fig. 4 Variation of *CSR* and $CRR_{7.5}$ (based on SPT values) values with depth for some typical boreholes

4.2 *CRR from CPT values*

In the recent studies, many simplified methods based on CPT have been developed for evaluating soil liquefaction potential (Shibata and Teparaksa, 1988; Youd and Idriss, 1997; Juang et al., 2003). Among these methods, the Youd and Idriss (1997) method is used more frequently.

The clean- sand base curve for determination of CRR based on corrected CPT tip resistance, (q_{c1N}) has been estimated by the following equation.

$$\begin{cases} if\,(q_{c1N}) < 50 \Rightarrow CRR_{7.5} = 0.833\left[\frac{(q_{c1N})}{1000}\right] + 0.05 \\ if\,50 < (q_{c1N}) < 160 \Rightarrow CRR_{7.5} = 93\left[\frac{(q_{c1N})}{1000}\right]^3 + 0.08 \end{cases} \quad (4)$$

where, (q_{c1N}) is the clean-sand cone penetration resistance, normalized to approximate 1 atmosphere (100 kPa) and corrected for thin layers. Plots of CSR and $CRR_{7.5}$ (based on CPT values) versus depth are presented in Fig. 5.

5 DISCUSSIONS

Based on the detailed liquefaction analysis, it was found that the upper layers of silty sand / fine sand are prone to liquefaction for an earthquake magnitude of 6.5 and peak horizontal ground acceleration of 0.16g, and for the water table at ground level.

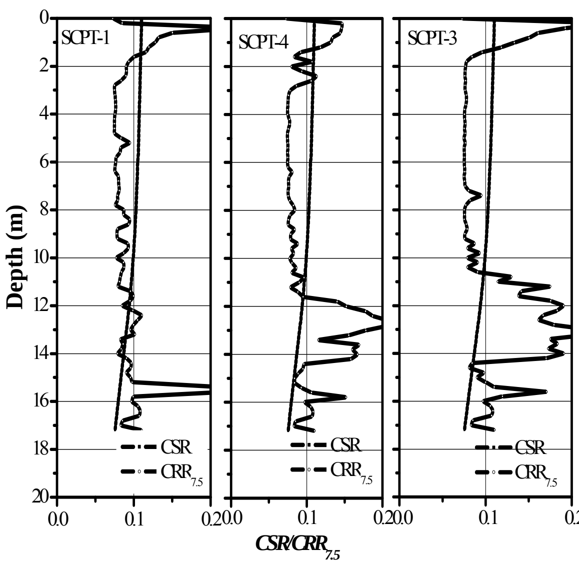

Fig. 5 Variation of *CSR* and $CRR_{7.5}$ (based on CPT values) values with depth for different SCPT.

The values of factor of safety for few typical boreholes are presented in Fig. 6. For the BH1 the factor of safety is close to 1 for depth of 2m to 12m whereas for BH1 the factor of safety is less than unity for the depth of 2.5m to 8m. Though various boreholes shows factor of safety less than unity at different elevations but in most of the cases it was found that the depth of liquefaction prone layer are within 5m to 12m from ground level.

The results from SCPT data show the same trend but the factor of safety calculated by CPT data is lower than the factor of safety calculated by SPT data.

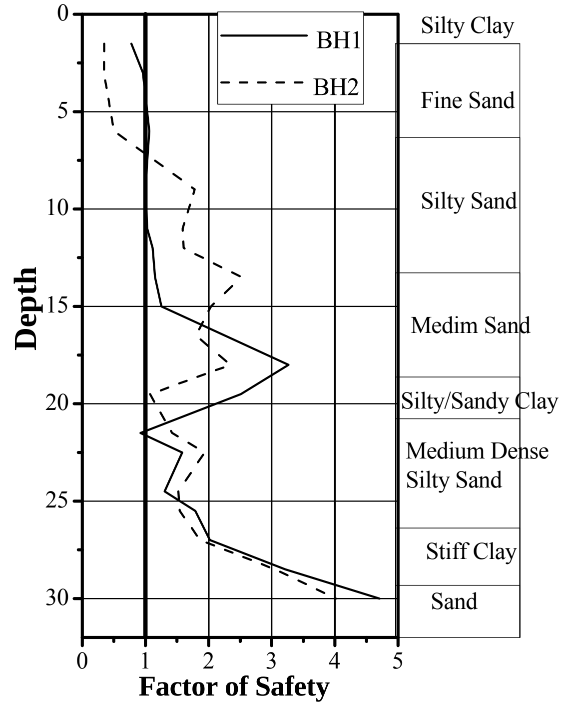

Fig. 6 Variation of factor of safety for some typical boreholes

EESL has recommended the use of 500mm to 600mm diameter bored cast-in-situ piles extending well below the liquefiable zone 23m to 28m, to transfer the loads to the more stable soil strata. Three initial load tests and two lateral load tests were conducted in order to access the capacity of pile foundation. As per the load test results obtained and as per IS:2911, the safe vertical capacity are well above 1400 kN and safe lateral capacity (without axial load applied on top of pile) are above 120 kN.

To raise level for safety against flooding during monsoon it was proposed to place the 3m fill of the dredged sand on the top of the existing ground level. These fill will provide extra overburden pressure and will enhance the factor of safety further as a result of net increase in effective overburden pressure. The results of the load tests on the recommended piles during the construction phase were in good agreement with static pile capacities calculated from initial load tests as presented in Fig. 7.

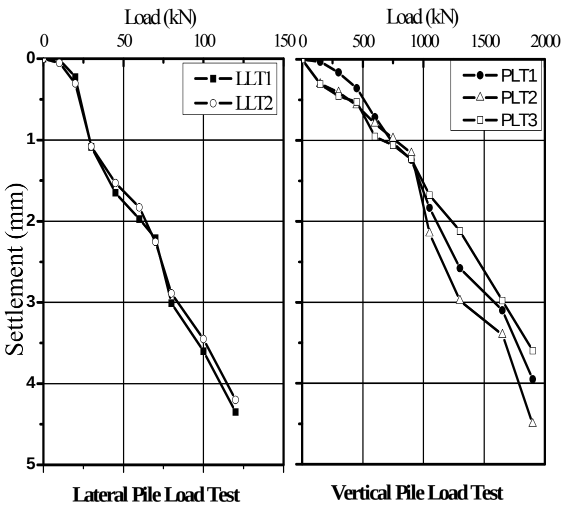

Fig. 7 Results of lateral and vertical pile load tests

6 CONCLUSIONS

In the present study, liquefaction potential assessment of Paradeep site has been carried out in order to select suitable foundation type for proposed Pellet plant. Stress-based simplified method is adopted and liquefaction analysis has been carried out using SPT and CPT field test data. Based on the study, it was observed that the upper layer of fine to medium silty sand within depth of 5 to 12m from ground level is prone to liquefy. The factor of safety from SPT and CPT are incomparable but both methods have indicated the possible liquefaction with in upper layer of silty sand. Based on the study, pile foundations extending upto 25m to 28m depth are adopted as suitable foundation system for entire project.

Based on the recommendation of IS:1893-2002, the contribution of liquefiable layer in the pile capacity was ignored and finally EESL recommended theoretical safe pile compressive, uplift and lateral capacities of 900 kN, 350 kN, 70 kN respectively, for 25m to 28 m long, 600 mm diameter bored cast-in-situ under seismic conditions.

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LIST OF NOTATIONS

τ_{av} = average horizontal shear stress acting on soil element during earthquake shaking
r_d = Stress reduction coefficient, based on Liao and Whitman (1986)
σ_{vo} = Total vertical overburden stress
σ_{vo}' = Effective vertical overburden stresses (based on design ground water depth of 0.0 m)
g = acceleration due to gravity
a_{max} = Peak horizontal ground accelerations (PGA)
z = Depth below ground surface, meters