Liquefaction Risk Mitigation for a Power Plant in the Indo-Gangetic Alluvium

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ABSTRACT: The paper presents the case study of liquefaction studies for an upcoming power plant in the Indo-Gangetic plain. Geotechnical site characterization studies suggested that the sands to 8 m depth were prone to liquefaction during the design earthquake. Accordingly, TG, chimney and other heavily-loaded facilities were supported on piles extending well below the liquefiable zone. Ground improvement by vibro-replacement was done in other areas of the site to mitigate the liquefaction potential.

1 INTRODUCTION

A 108 MW gas-based, combined cycle power plant is planned to be installed in North Delhi. The facilities in plant include the STG, GTG, steam turbine, water cooling system, switchyard, non-plant structures, etc.

A detailed geotechnical investigation was carried out at the site to assess the liquefaction susceptibility of the soils and to evaluate suitable foundation systems for each of the various plant facilities. The study indicated the presence of loose alluvial sands that were prone to liquefaction in the event of the design earthquake.

Heavily-loaded facilities were supported on piles. Ground improvement by vibro-replacement was done in other areas to mitigate the potential for liquefaction.

2 GENERAL SITE CONDITIONS

2.1 *Geological Setting*

The deposits in the area belong to the Indo-Gangetic Alluvium and are river deposits of the River Yamuna and its tributaries. The Pleistocene and Recent deposits in the project area are composed primarily of sands and silts.

The newer Alluvium, locally called *Khadar* (Krishnan, 1986), consists primarily of fine sand that is often loose in condition at shallow depths.

2.2 *Scope of Geotechnical Investigation*

The geotechnical investigation included 15 boreholes drilled to 30 m depth and 6 static cone penetration tests (CPT).

Spectral analysis of surface waves (SASW) was performed along eight lines in the Power Block area. These tests were conducted using Freedom NDT PC, along with pairs of geophones having natural frequencies of 1 Hz and 4.5 Hz. A 20 kg sledgehammer was used to generate the source.

Three cross-hole seismic tests (CHST) were conducted to 30 m depth to obtain the shear and compression wave velocity profile at the site.

A layout plan illustrating the location of the various field tests is presented on Fig. 1.

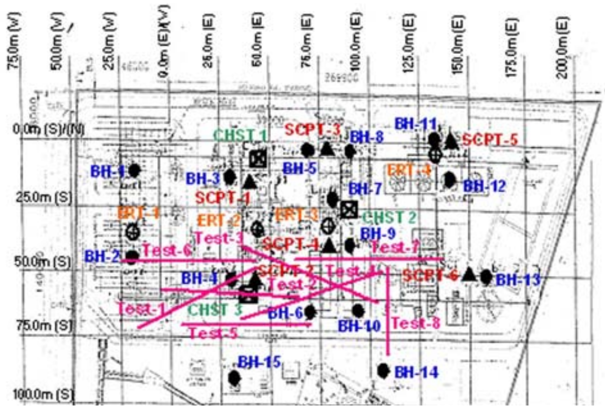

Fig. 1: Site Layout Plan

2.3 *Site Stratigraphy*

A surficial fill is present to about 0.5~2 m depth across the site. The natural soils below the fill consist of silty sand / fine sand with intermediate layers of sandy silt to 30 m depth. A profile of selected boreholes is illustrated on Fig. 2.

2.4 *Groundwater*

Groundwater was encountered at about 5.2-6.4 m depth (April-May 2008). Considering the seasonal variations, the design groundwater level was considered at the ground level.

2.5 *Field Tests*

Standard penetration tests (SPT) were performed using an automatic trip hammer. Low N-values ranging from 4 to 11 were encountered to about 10-12 m depth. Below this, the SPT values generally exceed 20-30.

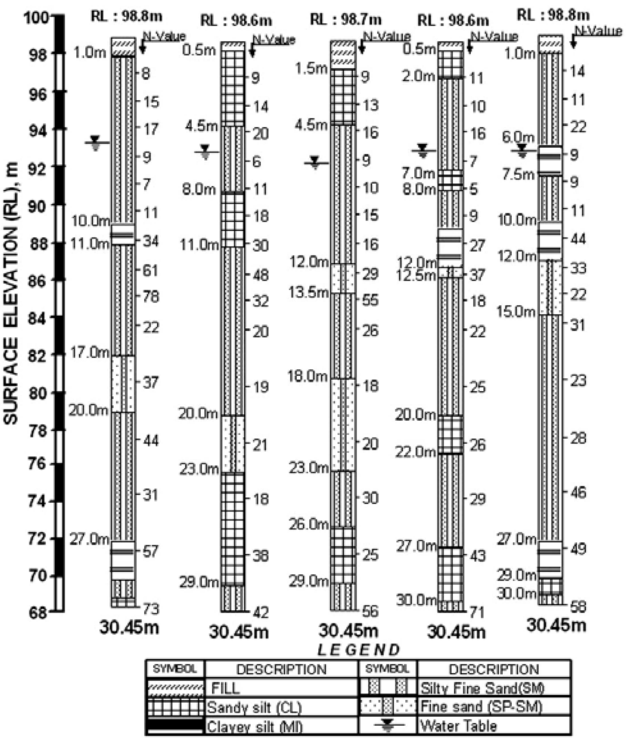

Fig. 2 : Site Stratigraphy

SCPT results showed a similar trend, and generally ranged from 0.5-4 MPa to about 8-10 m depth. Below this, SCPT cone tip resistance values increased to 8-16 MPa to about 14 m depth, underlain by lower values of 4-8 MPa, until refusal to further penetration was met at 14~18 m depth.

The measured shear wave velocities (V_s) generally ranged from 135-180 m/s to 5 m depth, 200-240 m/s to 10 m depth and 250-315 m/s to 30 m depth. Average shear wave velocities interpreted from the SASW tests were about 10~20% higher than the velocities interpreted from CHST tests. Both methods indicated the presence of soft soils (S_E as per UBC, 1997) to about 8-9 m depth.

Fig. 3 presents the shear wave velocity measurements based on SASW and CHST. Plots of measured SPT, cone tip resistance and shear wave velocity with depth, as well as respective design profiles used for design, are presented on Fig. 4.

3 LIQUEFACTION SUSCEPTIBILITY ANALYSIS

3.1 *Methodology and Design Parameters*

As per the seismic zoning map of India, Delhi is located in Earthquake Zone-IV, which places it in a high risk category.

The Indian Standard code (IS 1893, Part 1: 2002) suggests that liquefaction is likely in fine sands below water table with corrected SPT values (N') less than 15 to about 5 m depth and less than 25 below 10 m depth. According to this, there is a potential for liquefaction at the project site to about 10 m depth.

A detailed study was carried out for a better assessment of the liquefaction potential at the site. The methodology is based on the simplified procedure developed by Seed and Idriss (1971), as described in the NCEER Summary Report (2001). A design earthquake magnitude of 6.7 and peak ground acceleration (PGA) of 0.24 g were considered to represent the Maximum Credible Earthquake (MCE) at the project location.

3.2 *Cyclic Stress and Resistance Ratios*

The cyclic stress ratio (CSR) has been calculated for the selected PGA at various depths using the following equation-

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

where:
τ_{av} = Average horizontal shear stress acting on soil element during earthquake shaking
r_d = Stress reduction coefficient (Liao et. al., 1986)
σ_{vo} = Total vertical overburden stresses
σ_{vo}' = Effective vertical overburden stresses [*based on design groundwater depth of 0.0 m*]
g = acceleration due to gravity
a_{max} = Peak horizontal ground acceleration (PGA)
z = Depth below ground surface, meters

To avoid the difficulties and high costs associated with high-quality soil sampling and advanced laboratory testing at in-situ stress states for determination of cyclic resistance ratio (CRR), field tests have become the state-of-practice for routine investigations.

A Magnitude Scaling Factor (MSF) of 1.334 (based on Revised Idriss Scaling Factors recommended by the NCEER Summary Report, 2001) was applied to the CRR values, to adjust the clean sand curves to the design earthquake magnitude of 6.7.

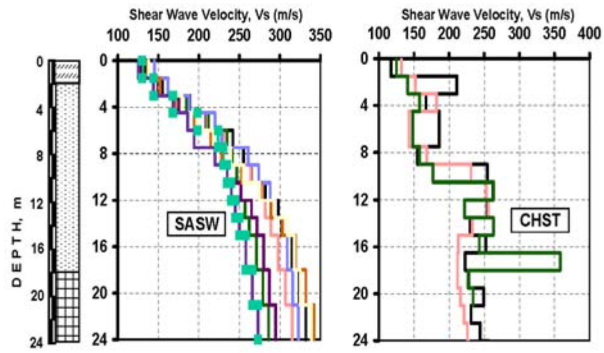

Fig. 3 : Shear Wave Velocity Tests

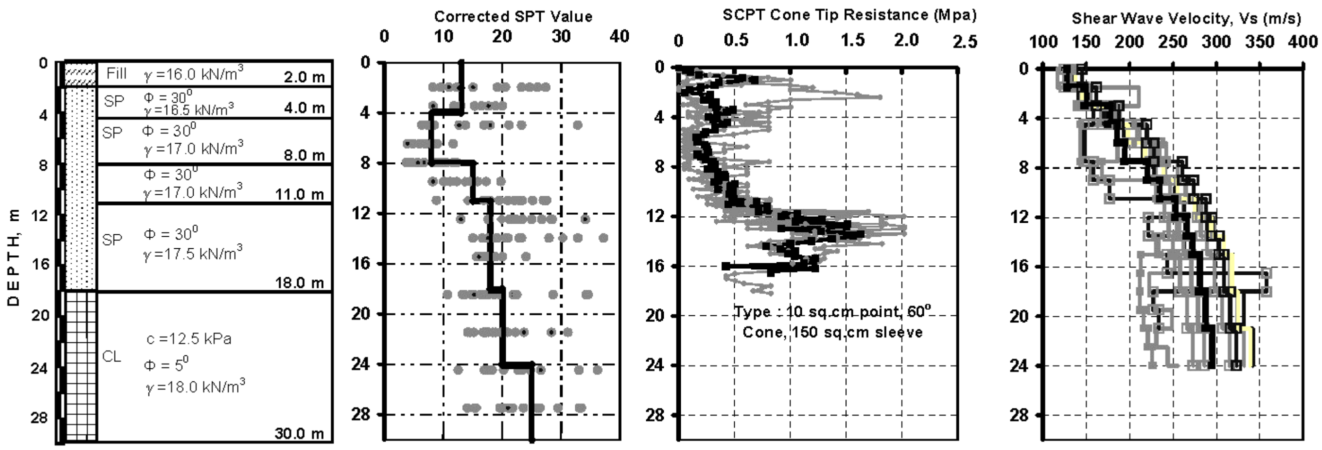

Fig. 4: Design Profile

The CRR profiles were computed based on SPT, CPT, and Vs values. The results of the liquefaction analysis are summarized on Fig. 5.

3.3 *Results*

Based on the project specifications, a critical factor of safety of 1.2 was considered for the analysis. Based on the detailed liquefaction analyses, the authors are of the opinion that the soils to a depth of 8.0 m may be susceptible to liquefaction in the event of the design earthquake.

4 FOUNDATION SYSTEM SELECTED

In view of the potential for liquefaction, open foundations bearing on natural soils are not a feasible foundation system. Appropriate foundation systems were selected for the various plant facilities, based on the loading conditions and permissible settlement criteria.

4.1 *Bored Piles*

For the critical or heavily-loaded plant facilities such as the TG, chimney, etc., it was decided to provide 600 mm-diameter bored cast-in-situ piles extending well below the liquefiable zone.

The safe (factor of safety of 2.5) pile compressive and uplift capacities for the 600 mm diameter 20 m long bored cast-in-situ pile with cut-off-level of 2 m is as follows:

Seismic Condition (liquefaction to 8 m depth):

Compression : 740 KN

Uplift : 420 KN

Normal Condition (no liquefaction)

Compression : 810 KN

Uplift : 480 KN

4.2 *Vibro-replacement*

For medium-loaded facilities such as the clariflocculator, cooling tower, etc., ground improvement was done by vibro-replacement method, to compact the loose soils and to limit the liquefaction potential (Sundaram and Gupta, 1994).

Dry vibro-stone columns of 500 mm-diameter with center to center spacing of 1.5 m were installed in these areas to 10 m depth by bottom feed method, as shown on Fig. 6.

The stone columns were designed for a safe net bearing pressure of 160 kPa.

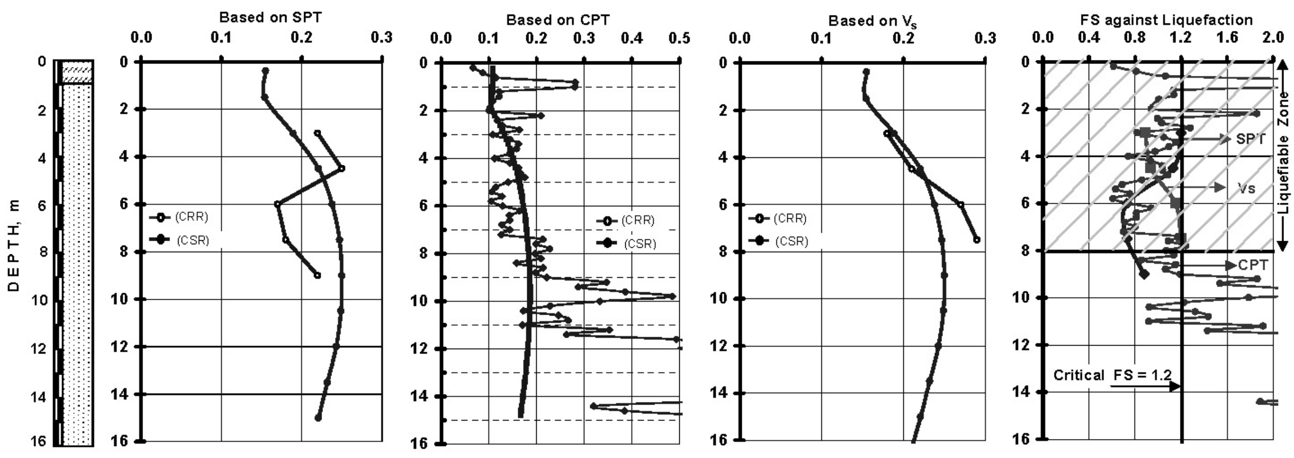

Fig. 5: Results of Liquefaction Analysis

Fig. 6 : Vibro-replacement in progress

Dynamic cone penetration tests (DPCT) were carried out on the improved ground using a 50 mm diameter cone driven by a 63.5 kg hammer falling through 75 cm as per IS:4968-1976 (Part 1). The DPCT results are presented on Fig. 7. DCPT blow counts (blows per 30 cm penetration) ranged from 15 to 25 in the top 4 m and 20 to 50 to 8 m depth. Evaluating the results, it was concluded that the improved ground is not likely to liquefy in the event of the design earthquake.

To assess the load-settlement characteristics of the improved ground, load tests were performed on the stone columns. A 300 mm thick compacted sand pad was placed over the stone columns. A 30 mm thick, 1.5 m x 1.5 m square test plate with steel stiffeners was placed over the sand pad (Fig. 8).

The single column load test was performed up to a loading intensity of 240 kPa in the first loading cycle, and 500 kPa in the second cycle.

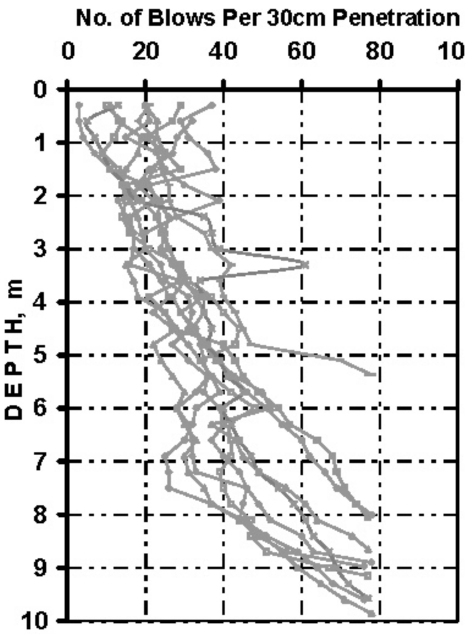

Fig. 7: Post-improvement DCPT Results

Fig.8 : Load Test on Stone column

The test results are presented on Fig. 9. Based on the test results, the authors' concluded that foundations being on the improved ground are safe for the design bearing pressure of 160 kPa.

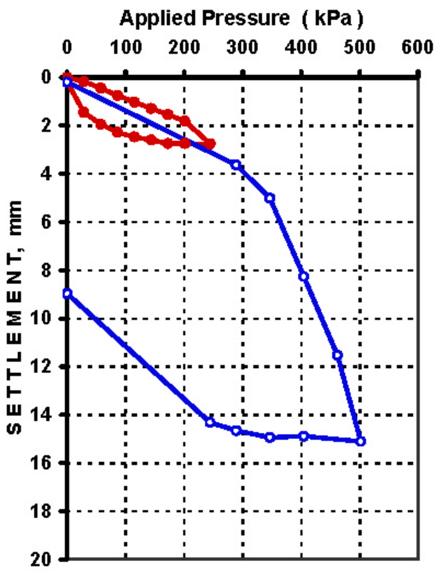

Fig. 9: Results of Load Test on Stone Column

5 CONCLUSIONS

This case study demonstrates the successful mitigation of the risk of liquefaction under the design earthquake. The detailed geotechnical investigation including shear wave velocity tests were used effectively to identify the liquefaction potential. Piling as well as ground improvement ensured that the power plant shall be safe and shall not be affected by the risk of liquefaction.

Liquefaction is a major consideration affecting foundation design in many parts of India. Often, the delicate balance of project costs, schedules and long-term success, is hinged on the geotechnical engineer's ability to predict, assess and deal with liquefaction susceptibility effectively.

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