LIQUEFACTION SUSCEPTIBILITY STUDY
AT COMMONWEALTH GAMES VILLAGE, DELHI

Sanjay Gupta, Ravi Sundaram, Sorabh Gupta
Cengrs Geotechnica Pvt. Ltd., A-100, Sector 63, Noida-201309 (UP), cengrs@gmail.com

G.V. Ramana, G. Masthan, Rana Sen and K. Surya Narayana
Department of Civil Engineering, Indian Institute of Technology, Delhi

ABSTRACT: Geotechnical investigations were carried out at the site of the Commonwealth Games-2010 Village site near Akshardham in New Delhi. The study included Spectral analysis of Surface Waves (SASW) tests conducted in conjunction with boreholes and static cone penetration tests to assess the liquefaction susceptibility during earthquakes. Detailed liquefaction susceptibility studies confirmed that soils to a depth of about 9.5 m are susceptible to liquefaction hazard. The paper discusses the approach used for the analysis together with the foundation system adopted.

INTRODUCTION

The Commonwealth Games-2010 Village is being constructed in the heart of Delhi near I.P Estate and the Akshardham Temple on a 40-acre land. The Village shall accommodate 8,500 athletes and officials during the 2010 Commonwealth Games to be held in Delhi. Figure 1 presents an overview of the site.

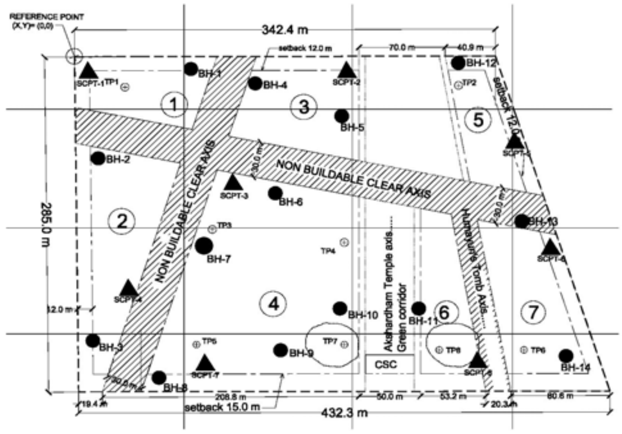

Fig. 1. Site Layout Plan

The structures planned on site include a multi-storied housing complex with stilts, and upper floors ranging from 5 to 9 storeys in different towers. A single basement is planned in the tower blocks, while the non-tower blocks shall be without basement. The site is fairly leveled with ground levels ranging from RL 201.9 m to RL 202.8 m at various test location. The average ground level is fixed as RL 202.5 m as per architectural plans.

GENERAL SITE CONDITIONS

Geological Setting

The project site is in East Delhi adjoining the Yamuna riverbed. The soils at the project site belong to the "*Indo Gangetic Alluvium*" and are river deposits of the Yamuna and its tributaries. The Pleistocene and Recent Deposits of the Indo-Gangetic Basin (Krishnan, 1986) are composed of gravels, sands, silts and clays. The newer alluvium, deposited in the areas close to the river, is locally called "*Khadar*" and consists primarily of fine sand that is often loose in condition to about 4-10 m below the ground surface.

Scope of Investigation

The scope of the geotechnical investigation included fourteen boreholes to 30 m depth and eight static cone penetrometer tests (SCPT). In addition, 23 shear wave velocity tests are conducted using Spectral Analysis of Surface Waves (SASW) technique along six spreads across the site.

Site Stratigraphy

The surficial soils at the site consist primarily of sandy silt / clayey silt to about 0.5 to 2.0 m depth. This is underlain by sand / silty sand to the final explored depth of 30.45 m. In general, the sands are fine-grained with fines content ranging from 4 to 12 percent. A discontinuous zone of hard clayey silt / sandy silt varying in thickness from 2 to 8 m is met below 15-18 m depth at some borehole locations. A summary of selected borehole profiles is illustrated on Figure 2.

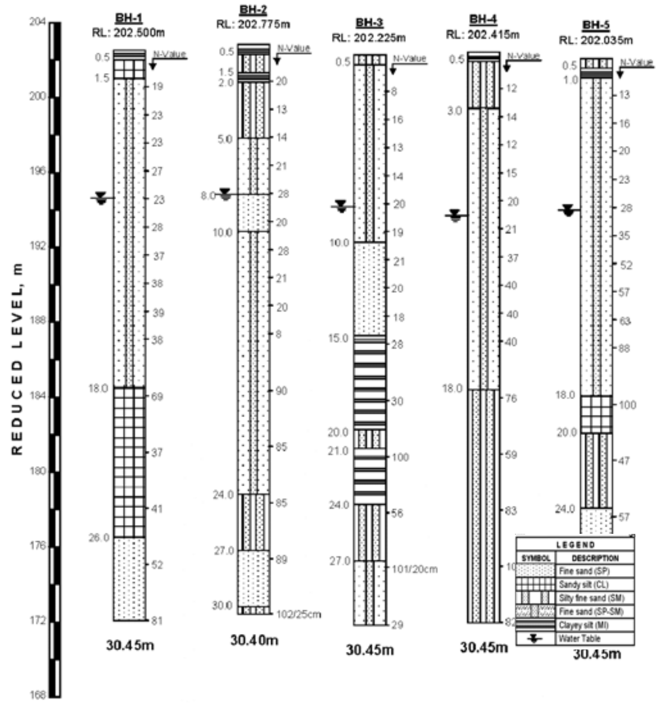

Fig. 2. Site Stratigraphy

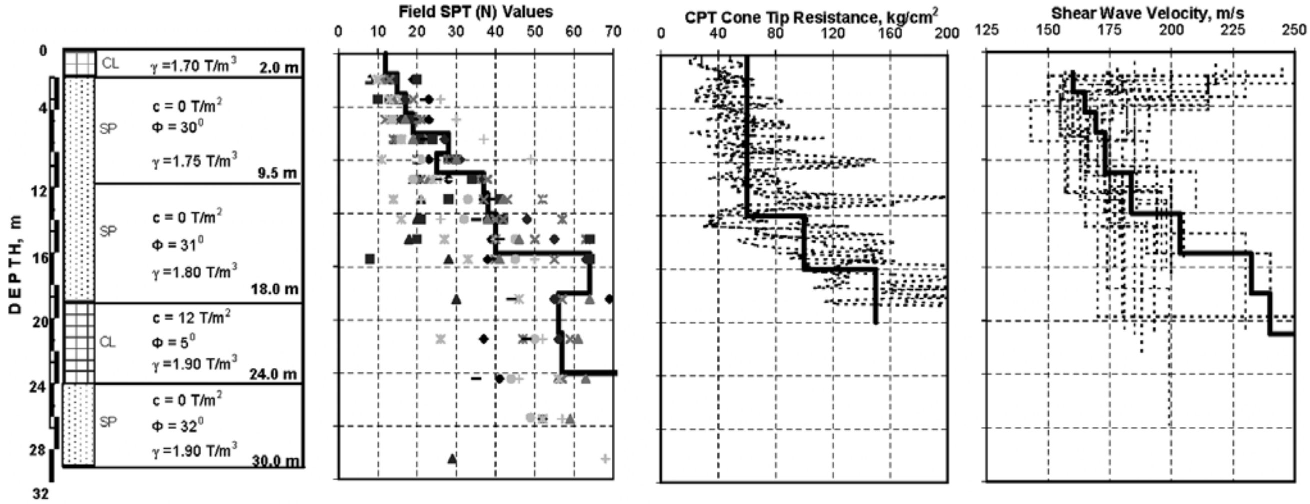

Fig. 3. Design Soil Profiles

Field Tests

SPT was performed using an automatic trip hammer. Field SPT values range from 8 to 20 to about 5.0 m depth and from 15 to 30 to about 8.0 m depth. Below this, SPT values increase with depth, ranging from 25 to 40 at 10.0 m depth to 50 to 90 at 18.0 m depth. In the underlying soils, SPT values range from 50 to refusal ($N \geq 100$) to the maximum explored depth of 30 m.

SCPT results indicate cone tip resistances (q_c) of 35 to 70 kg/cm^2 to 4.0 m depth, and 40 to 80 kg/cm^2 to about 9.5 m depth. Below this, q_c values range from 60 to 160 kg/cm^2 to 12.0 m depth, 60 to 130 kg/cm^2 to 15 m depth, 100 to 200 kg/cm^2 to 17 m depth, and 120 to 250 kg/cm^2 to the maximum depth of 20 m.

The shear wave velocity at the site was measured using SASW tests conducted along six lines across the site. These tests were conducted using Freedom NDT PC, along with pairs of geophones having natural frequencies of 1 Hz and 4.5 Hz. A sledgehammer (20 kg) was used to generate the source.

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 Figure 3.

Groundwater

As measured in the completed boreholes, groundwater was met at about 7.1 to 8.8 m depth (July 2007). Considering that the site is in the vicinity of the Yamuna River, the design groundwater table has been considered at the ground surface.

ASSESSMENT OF LIQUEFACTION SUSCEPTIBILITY

Basic Concept

Liquefaction is defined as the transformation of a granular material from a solid to a liquefied state as a consequence of increased pore-water pressure and reduced effective stress (Marcuson, 1978). Increased pore pressure may be induced by the tendency of granular materials to compact when subjected to cyclic shear deformation, such as in the event of an earthquake.

Methodology and Design Parameters

As per IS:1893 (Part-1):2002, Table-1 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). For values of depths between 5 m to 10 m, linear interpolation is recommended. As per the IS Code guidelines, there is a potential for liquefaction of the soils to about 14.0~16.0 m depth.

Detailed liquefaction analysis has been carried out for the site. The methodology is based on the simplified procedure developed by Seed and Idriss (1971), as described in the NCEER Summary Report (2001). As per the project specifications, the analysis has been carried out for design earthquake magnitudes of 6.7 and peak ground accelerations of 0.24 g.

Cyclic Stress Ratio

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-

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

and

$$\begin{cases} r_d = 1.0 - 0.00765z \text{ for } z \leq 9.15\,\text{m} \\ r_d = 1.174 - 0.0267z \text{ for } 9.15 \leq z \leq 23\,\text{m} \end{cases} \tag{2}$$

where:

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 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

Cyclic Resistance Ratio

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 CRR, field tests have become the state-of-practice for routine liquefaction investigations.

The cyclic resistance ratio (CRR) at the site has been computed based on SPT values, CPT values, and Shear Wave Velocity (V_s) values.

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.

CRR from SPT Values

Since SPT N-values increase with increasing effective overburden pressure, an overburden stress correction factor is applied (Seed & Idriss 1982) to normalize N_{60} to an equivalent effective overburden pressure of 1 atmosphere (100 kPa). The overburden correction on SPT values is based on actual depth of groundwater, as measured in the field. A correction for hammer energy ratio (ER) of 0.75 has been applied to the SPT values. Other correction factors for borehole diameter, rod length, and samplers with or without liners have also been taken into account. The SPT values have further been corrected for fines content, as per the equations suggested by Seed & Idriss (1971).

The clean-sand base curve for determination of CRR based on corrected SPT N-values, $(N_1)_{60}$, has been estimated by the following equation (Rauch, 1998) for $(N_1)_{60} \leq 30$:

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

Plots of field and corrected SPT values, as well as CSR and CRR (based on median SPT values) versus depth, are presented on Figure 4.

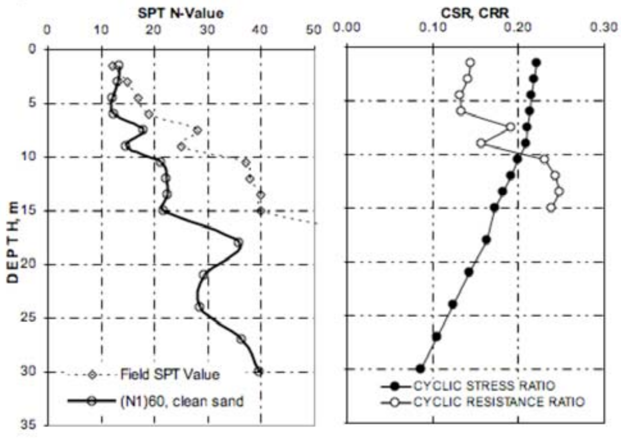

Fig. 4. SPT-based Liquefaction Curves

CRR from CPT Values

The clean-sand base curve for determination of CRR based on corrected CPT tip resistance, (q_{c1N}) has been estimated by the following equation (Robertson & Wride, 1998):

$$\begin{cases} \text{If } (q_{c1N})_{cs} < 50 \Rightarrow CRR_{7.5} = 0.833\left[{(q_{c1N})_{cs}}/{1000}\right] + 0.05 \\ \text{If } 50 \leq (q_{c1N})_{cs} < 160 \Rightarrow CRR_{7.5} = 93\left[{(q_{c1N})_{cs}}/{1000}\right]^3 + 0.08 \end{cases} \tag{4}$$

[$(q_{c1N})_{cs}$ is the clean-sand cone penetration resistance, normalized to approx. 1 atmosphere (100 kPa) and corrected for thin layers]

Plots of field and corrected cone tip values, as well as CSR and CRR (based on median q_c values) versus depth, are presented on Figure 5.

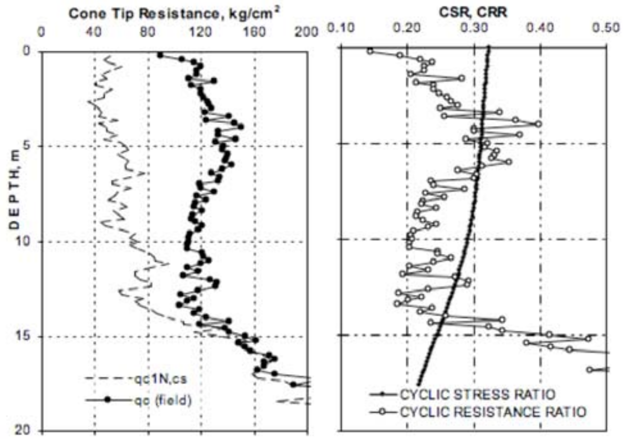

Fig. 5. CPT-based Liquefaction Curves

CRR from Shear Wave Velocity (V_s) values

The use of V_s as a field index of liquefaction resistance is a sound approach because both V_s and CRR are similarly, but not proportionally, influenced by void ratio, effective confining stresses, stress history, and geologic age.

A design shear wave velocity profile was developed for analysis based on median V_s values (Fig. 3), after neglecting data that had a large deviation from the median. The V_s values were corrected to a reference overburden stress using Equation 5 (Sykora 1987, Robertson et. al. 1992) to obtain the overburden-stress corrected shear wave velocity, V_{s1} (Fig. 6).

$$V_{s1} = V_s \left(\frac{P_a}{\sigma'_{vo}} \right)^{0.25} \tag{5}$$

CRR was computed based on CRR versus V_{S1} curves recommended for engineering practice by Andrus & Stokoe (1997, 2000, 2004) for magnitude 7.5 earthquakes and Holocene-age soils with various fines contents; and estimated using the following relationship (Andrus & Stokoe, 1997):

$$CRR = a\left(\frac{V_{s1}}{100}\right)^2 + b\left(\frac{1}{V^*_{s1} - V_{s1}} - \frac{1}{V^*_{s1}}\right) \tag{6}$$

[V_{s1}^* is the limiting value of V_{s1} for liquefaction occurrence]

Profiles of field and corrected shear wave velocity, CSR and CRR (based on design V_s values), are presented on Figure 6.

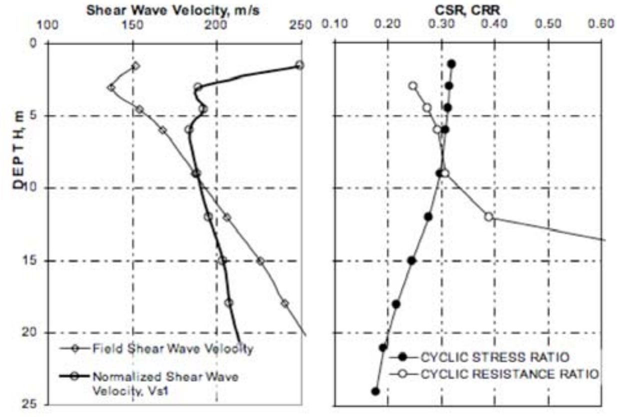

Fig. 6. V_s-based Liquefaction Curves

RESULTS OF ANALYSIS

Based on the detailed liquefaction analyses, the authors are of the opinion that soils to a depth of 9.5 m below original ground levels are prone to liquefaction susceptibility for an earthquake magnitude of 6.7, a peak horizontal ground acceleration of 0.24 g, and for a design groundwater table at ground level.

FOUNDATION SYSTEM SELECTED

In view of the potential for liquefaction, open foundations bearing on natural soils are not a feasible foundation system. The authors recommended the use of 500 mm- to 1200 mm-diameter bored cast-in-situ piles extending well below the liquefiable zone, to transfer the loads to the more stable soil strata.

The authors recommended theoretical safe pile compressive and uplift capacities of 126 Tonnes and 70 Tonnes, respectively, for 20 m long, 800 mm diameter bored cast-in-situ piles with a pile cut-off-level (COL) of 2.0 m under seismic conditions.

PILE LOAD TESTS

Six vertical and six lateral initial pile load tests were performed at the site on 500, 750 and 800 mm diameter piles of 20~22 m length below COL to assess the safe load-carrying capacity of the piles. A 600-tonne capacity load frame (Figure 7) was used to carry out the tests on site.

The results of the load tests were in good agreement with theoretical static pile capacities calculated from soil parameters for the normal (no liquefaction) condition. Final pile capacities to be used for design were then recommended based on the above computations, taking liquefaction into account under seismic conditions.

Fig. 7. Pile Load Test Setup

CLOSURE

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.

A major thrust in this direction is required from academia and industry alike to standardize and raise the industry standard in this respect, so that future disasters may be averted by the use of safe and economical design practices.

REFERENCES

Andrus, R.D., Stokoe II, K.H. and Juang, C.H. (2004), "Guide for Shear-Wave-Based Liquefaction Potential Evaluation", Earthquake Spectra, Volume 20, No. 2, May 2004, 285-308

Andrus, R.D., Stokoe, K.H., II.,(1997), "Liquefaction Resistance based on shear wave velocity", Proc., NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, Nat. Ctr. for Earthquake Engrg. Res. State Univ. of New York at Buffalo, 89-128.

Andrus, R.D., Stokoe, K.H., II.,(2000), "Liquefaction Resistance of soils from shear-wave velocity.", J. Geotech. And Geoenvir. Engrg., ASCE, 126(11), 1015-1025.

IS:1893-2002 (Part 1): Indian Standard for Criteria for Earthquake Resistant Design of Structures – General Provisions and Buildings, Bureau of Indian Standards, New Delhi

Krishnan, M.S. (1986), *Geology of India & Burma*. CBS Publishers, New Delhi.

Liao, S.S.C., and Whitman, R.V. (1986), "Catalogue of liquefaction and non-liquefaction occurrences during earthquakes", Res. Rep., Dept. of Civil Engrg., MIT, Mass.

Markuson, W.F. (III) (1978), "Definition of terms related to liquefaction", J. Geotech Engrg. Div,, ASCE, 104(9), 1197-1200

Rauch, A. F. (1998). Personal Communication. (as cited in Youd, et. al. (2001)).

Robertson, P.K., Woeller, D.J., and Finn, W.D.L. (1992), "Seismic Cone Penetration Test for evaluating liquefaction potential under cyclic loading", Can. Geotech. J. 29(4), 686-695

Roberson, P.K., and Wride, C.E. (1998), "Evaluating cyclic liquefaction potential using the cone penetration test", Can. Geotech. J., Ottawa, 35(3), 442-459

Seed, H.B., and Idriss, I.M (1971), "Simplified Procedure for evaluating soil liquefaction potential.". J. Geotech. Engrg. Div., ASCE, 97(9), 1249-1273

Seed, H.B., and Idriss, I.M. (1982), "Ground motions and soil liquefaction during earthquakes.", Earthquake Engineering Research Institute Monograph, Oakland, Calif.

Sykora, D.W., 1987, "Creation of a data base of seismic shear wave velocities for correlation analysis.", Geotech. Lab. Misc. Paper GL-87-26, U.S. Army Engineers Waterways Experiment Station, Vicksburg, MS

'NCEER Summary Report (2001)': Youd, T. L., Idriss, I. M., Andrus, R. D., Arango, I., Castro, G., Christian, J. T., Dobry, R., Finn, W. D. L., Harder, L. F., Hynes, M. E., Ishihara, K., Koester, J. P., Liao, S. S. C., Marcuson, W. F., Martin, G. R., Mitchell, J.K., Moriwaki, Y., Power, M. S., Robertson, P. K., Seed, R. B., and Stokoe, K. H. (2001). "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 127(10), 817-833

Youd, T.L., and Idriss, I.M., (2001), "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils", Journal of Geotechnical and Geo-environmental Engineering, April 2001, 297-313.