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Commonwealth games village, Delhi – Liquefaction study

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1 INTRODUCTION

The Commonwealth Games -2010 were held in New Delhi, India in October 2010. The various sporting events were organized in different venues / stadiums all over the city. Sportspersons from over 70 countries participated in the Games that lasted for over 10 days.

The event was inaugurated at the Jawaharlal Nehru Stadium by Prince Charles of Great Britain in the presence of the Indian President, Her Excellency Mrs. Pratibha Patil and Prime Minister, Dr. Manmohan Singh.

To accommodate over 8500 athletes and officials during the event, the Commonwealth Games-2010 Village were constructed in the heart of Delhi (India) near I.P Estate and the Akshardham Temple on a 40-acre land. It is on the east bank of the River Yamuna. Fig. 1 presents the location of the village and Fig. 2 presents the Master Plan of the village complex.

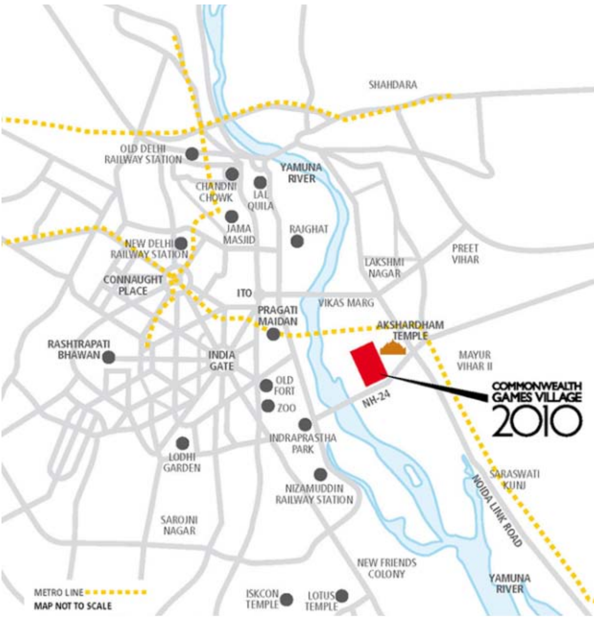

Figure 1 Site Location

Buildings Constructed

The structures constructed include a multi-storied housing complex with stilts, and upper floors ranging from 5 to 9 storeys in different towers. A total of 34 towers were constructed. A single basement is planned in the tower blocks, while the non-tower blocks are without basement. Badminton courts, a swimming pool, golf greens, promenade and food plaza, landscaped gardens, beautiful fountains, water bodies, etc. add to the elegance and charm of the area.

The site is fairly level with ground levels ranging from RL 201.9 m to RL 202.8 m. The average ground level is fixed as RL 202.5 m as per architectural plans.

2 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, Yamuna Sand, that is often loose in condition to about 4-10 m below the ground surface.

Scope of Geotechnical Investigation

The scope of the geotechnical investigation included 14 boreholes to 30 m depth and 8 static cone penetrometer tests (SCPT). Also, 23 shear wave velocity tests were conducted using Spectral Analysis of

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Surface Waves (SASW) technique along six spreads across the site. Fig. 3 presents the locations of the investigation program.

Figure 2 Master Plan of the Complex

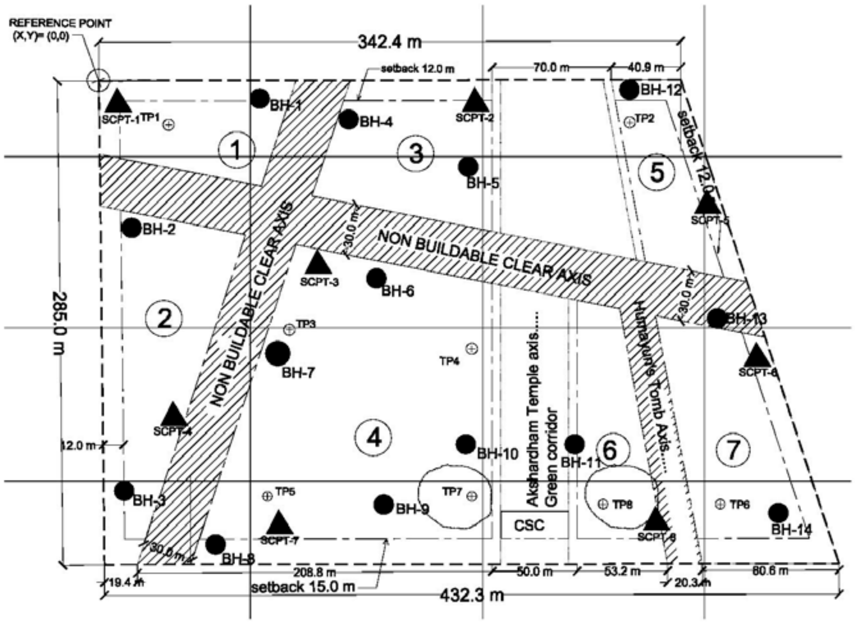

Figure 3 Plan of Field Investigation

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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 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 in Fig. 4.

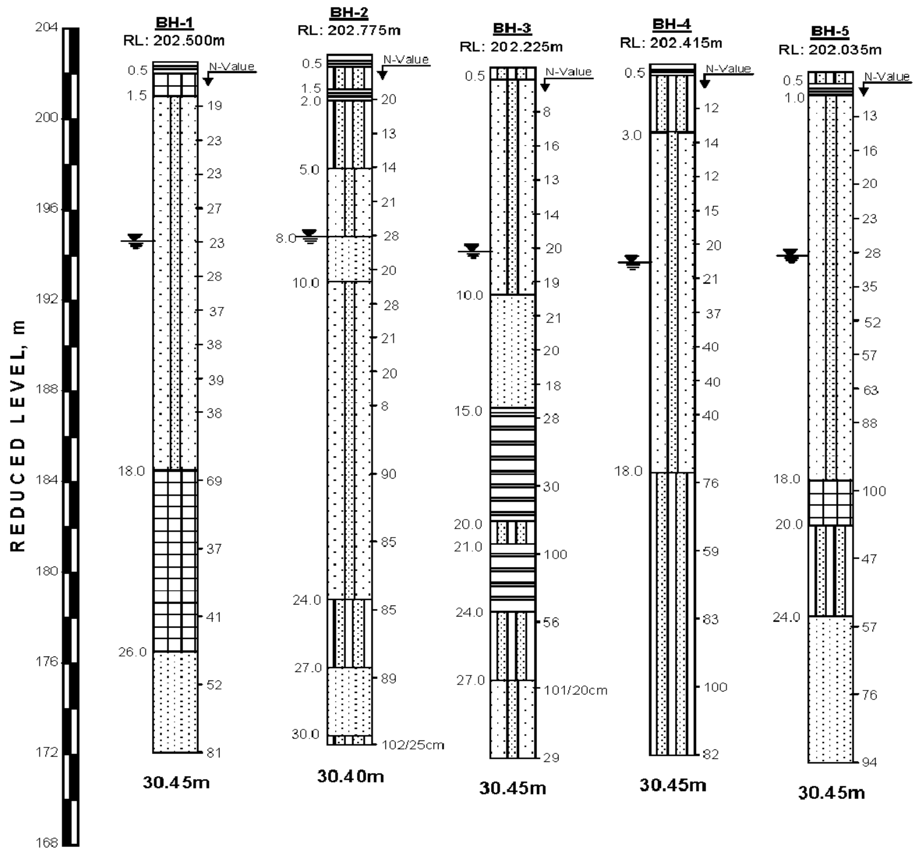

Figure 4 Site Stratigraphy

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 ≥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 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 in Fig. 5.

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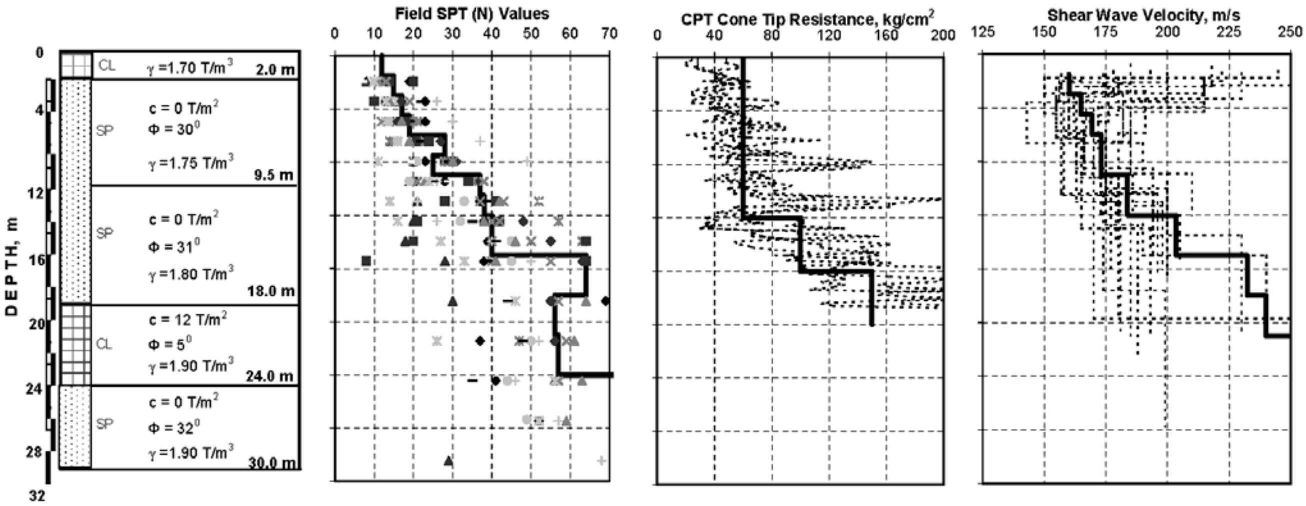

Figure 5 Design Soil Profile

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 were considered at the ground surface.

3 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 Indian Standard Code 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 were 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 were carried out for design earthquake magnitudes of 6.7 and peak ground accelerations of 0.24 g.

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Cyclic Stress Ratio

In this analysis, the cyclic stress ratio (CSR) were calculated for the selected peak horizontal ground accelerations at various depths using the following equations-

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

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

The cyclic resistance ratio (CRR) at the site were 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.

Plots of field and corrected SPT values, as well as CSR and CRR (based on median SPT values) versus depth, are presented in Fig. 6. Plots of field and corrected cone tip values, as well as CSR and CRR (based on median q_c values) versus depth, are presented in Fig. 7. Profiles of field and corrected shear wave velocity, CSR and CRR (based on design V_s values), are presented in Fig. 8.

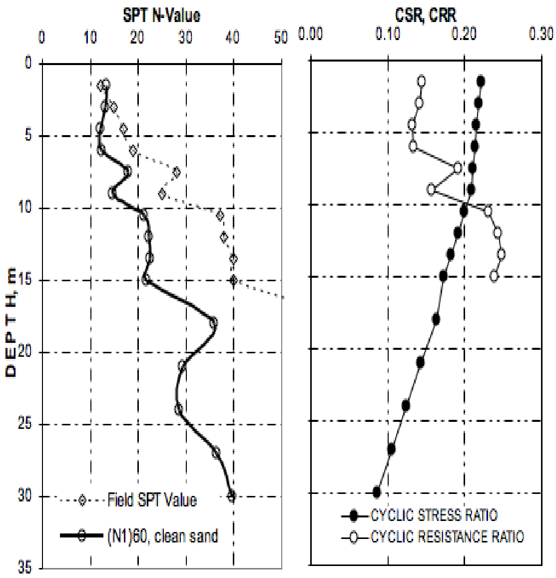

Figure 6 SPT-based Liquefaction Curves

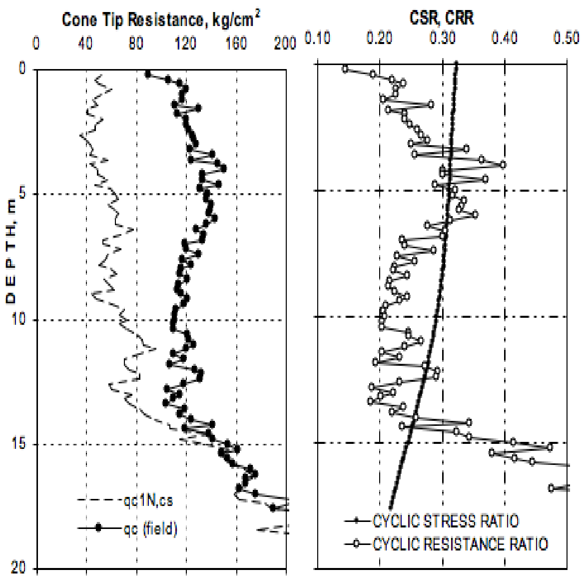

Figure 7 CPT-based Liquefaction Curves

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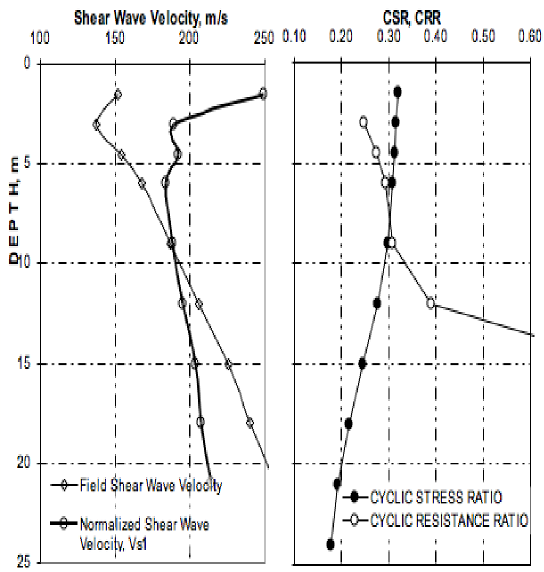

Figure 8 Vs-based Liquefaction Curves

4 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.

5 FOUNDATION SYSTEM SELECTED

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

Based on static analysis, 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.

6 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 (Fig. 9) 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.

Figure 10 is a photograph of the piling work in progress at the Commonwealth Games Village site. Fig. 11 presents a view of the constructed Commonwealth Games Village which was appreciated by the athletes and officials alike from over 70 countries. The Games was a huge success.

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Figure 9 Pile Load Test Setup
(See the Akshardham Temple in the background)

Figure 10 Piling in Progress at Site

Figure 11 A View of the Commonwealth Games Village Complex

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7 CONCLUSION

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 hinges 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.

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