IN-SITU TESTING TO VERIFY GROUND IMPROVEMENT

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ABSTRACT

Reliability of improvement achieved is the key to the implementation of soil stabilization and ground improvement measures. In-situ tests are an essential part of the ground improvement process to confirm that the desired improvement in soil properties has been achieved. The paper presents two case studies in the Delhi-NCR area where ground improvement was done to mitigate liquefaction potential and improve the soil bearing capacity. In-situ tests conducted before and after improvement to verify the effectiveness of improvement include SPT (in boreholes), static cone penetration tests (SCPT) and plate load tests.

INTRODUCTION

Ground improvement may be required to modify some of the soil properties to design the structure economically and as per the project requirement. It is usually done to densify loose soils, mitigate potential for liquefaction during earthquakes, to strengthen soft clays, to improve bearing capacity and to reduce foundation settlement.

The success of any ground improvement process depends on the effectiveness of the method in achieving the desired improvement. To do this, the target soil properties to be achieved should be clearly defined. This highlights the need for performing sufficient in-situ tests to verify that the soil properties have equalled or exceeded the target values.

Two case studies are presented here to demonstrate the effective use of in-situ tests to ensure successful improvement. The first case is of a Power Plant in north-Delhi where liquefaction potential was mitigated by installation of vibro-stone columns. The second case is of a large university campus in Greater Noida where dynamic compaction was performed to densify loose sands and mitigate liquefaction potential. In both cases, in-situ tests were conducted before and after the improvement to ensure that the desired extent of improvement is achieved.

CASE STUDY - 1

Project Details

Detailed geotechnical investigation carried out at the site of a 108 MW gas-based combined cycle power plant in north-Delhi 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. The facilities include the STG, GTG, steam turbine, water cooling system, switchyard, non-plant structures, etc.

A layout plan illustrating the location of the various field tests - 15 boreholes, 6 static cone penetration tests (CPT), 3 cross-hole seismic tests (CHST) and Spectral analysis of surface waves (SASW) tests along eight lines - is presented on Fig. 1.

Fig. 1: Site Layout Plan

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. The cone tip resistances are plotted on Fig. 3.

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

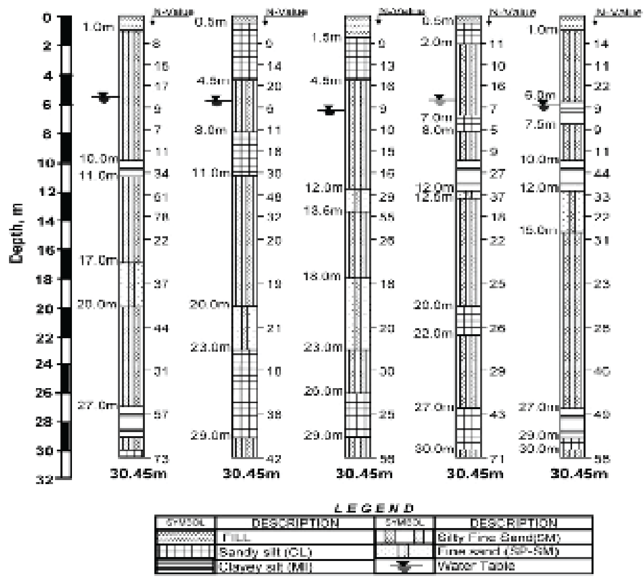


Fig. 2: Typical Borehole Profiles

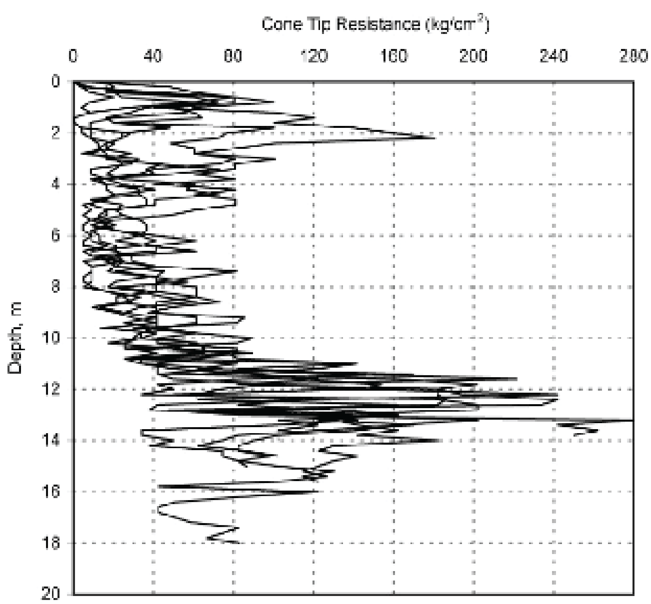


Fig. 3: Static Cone Penetration Test Results

Shear wave velocities from the SASW tests and CHST are presented on Fig.4.

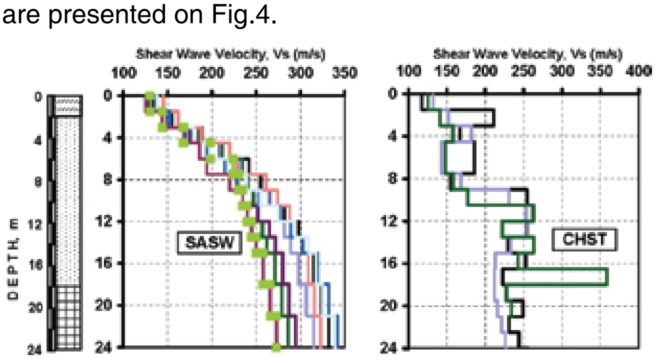


Fig. 4: Shear Wave Velocities from SASW and CHST

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. 5.

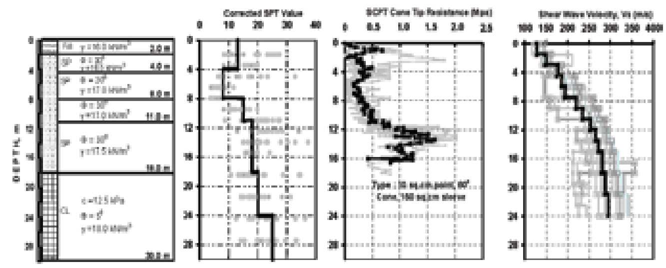


Fig. 5: Design Profile

Liquefaction Assessment

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). The project area falls in Earthquake Zone IV. Accordingly, a design earthquake magnitude of 6.7 on the Richter scale and peak ground acceleration (PGA) of 0.24 g were considered to represent the Maximum Credible Earthquake (MCE).

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 cyclic resistance ratio (CRR) values, to adjust the clean sand curves to the design earthquake magnitude of 6.7. The CRR profiles were computed based on SPT, CPT, and V_s values. The results of the liquefaction analysis are summarized on Fig. 6.

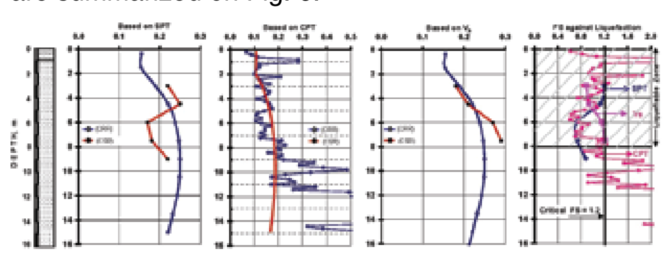


Fig. 6: Results of Liquefaction Analysis

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.

Foundation System

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.

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

Fig. 7: Vibro-Replacement in progress

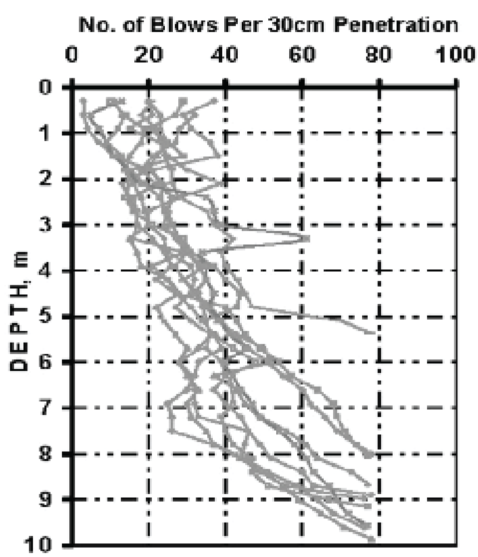


Fig. 8: Post-improvement DCPT Results

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. 9).

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. The test results are presented on Fig. 10. Based on the test results, the authors' concluded that foundations bearing on the improved ground are safe for the design bearing pressure of 160 kPa.

Fig. 9: Load Test on Stone column

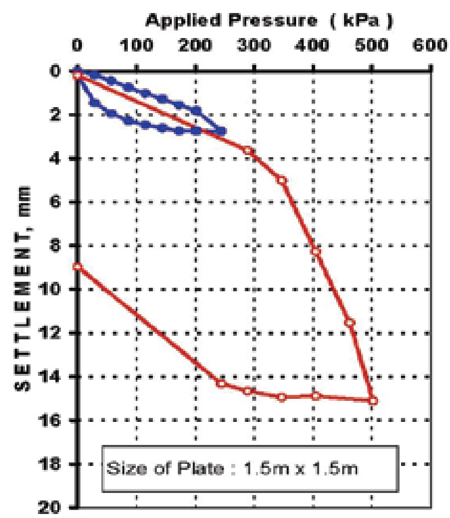


Fig. 10: Results of Load Test on Stone Column

CASE STUDY - 2

Project Details

A major University project spanning an area of over 500 acres is coming up at Greater Noida, Uttar Pradesh. The campus has over 84,000 m^2 of constructed area with 30 percent green cover. The site is located in the Yamuna flood plain, about 2 km. from the river (Fig. 11).

Geotechnical investigations carried out at the project site indicated the presence of liquefiable soils to about 8-12 m depth. The geotechnical investigation program executed at the project site includes drilling of over 600 boreholes and conducting more than 150 static cone penetration tests (SCPT) at the various structure locations. In order to mitigate the risk of liquefaction, ground improvement by densification of the loose sand was necessary.

Fig.11: Site Vicinity Map

The depth of liquefiable soils at each structure location was assessed based on Standard Penetration Test (SPT) and SCPT data. Boreholes and SCPT were also carried out at the site after dynamic compaction to confirm the extent of improvement achieved and to provide data for foundation analysis.

Details of ground improvement by dynamic compaction adopted for one of the Boys Hostel building, together with in-situ test results before and after the improvement are presented here. Four boreholes of 15 m depth and one SCPT were conducted at this structure location before and after dynamic compaction, as illustrated on Fig. 12.

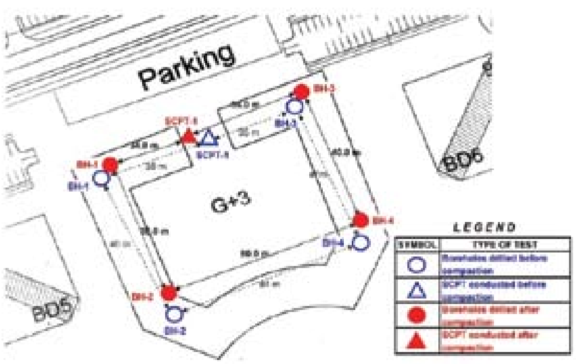


Fig. 12: Layout Plan – Boys Hostel No. 8.1K

Stratigraphy

The soils at the site classify primarily as sandy silt / clayey silt to about 2~3 m depth, underlain by fine sand to about 15 m depth (Fig. 13). The fines content of the sand stratum ranged from 5 to 10 percent. Groundwater was encountered at about 4~5 m depth. The design groundwater level was considered at the ground level.

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.

As per the liquefaction analysis using the NCEER approach, the authors estimated that the soils to a depth of 8 m may be susceptible to liquefaction in the event of the design earthquake at the Boys Hostel building.

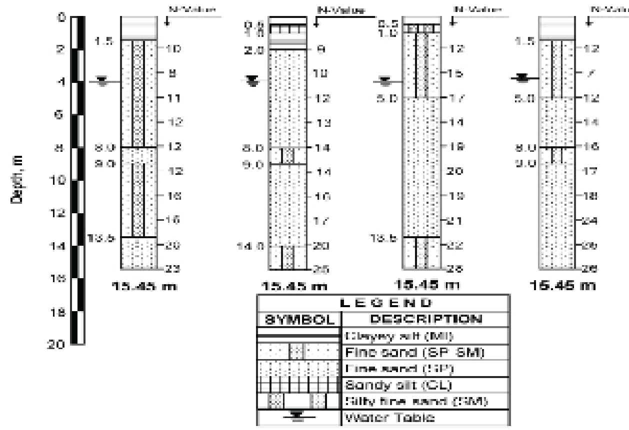


Fig. 13: Typical Borehole Profiles

Ground Improvement

Out of the various methods available for densification of cohesionless soils, dynamic compaction was adopted at the project site. Initial field trials were carried out in the university areas, which demonstrated substantial improvement. Figure 14 shows the conventional crane (TLC 955A) and the pounder used for performing the dynamic compaction.

In dynamic compaction, a pounder is repeatedly dropped on the ground surface from a specified height. The shock waves and high stresses induced by dropping the pounder result in the soil being compressed, together with partial liquefaction of the soil and the creation of preferential drainage paths through which pore water can be dissipated (see conceptual illustration on Fig.15). Several drops or poundings carried out in phases are required to achieve the desired improvement.

At the Boys Hostel building site, the contractor used a 114.3 kN pounder falling from a height of 14 m. The dynamic compaction was done in two phases, followed by an ironing phase. Details are explained below:

Fig. 14: Dynamic Compaction in progress on site

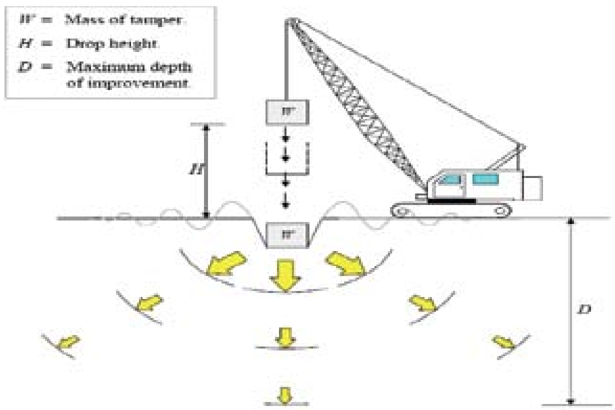


Fig. 15: Conceptual illustration of deep dynamic compaction

1. At each point on a 4 m x 4 m grid, a pounder of 114.3 kN was dropped repeatedly from a height of 14 m, i.e. an input energy of 1600 kN.m was applied. Usually, 10 or more blows were applied at each point and craters of about 1.0 to 1.5 m depth were formed. The volume of the craters was measured and recorded before they were backfilled with GSB Grade II.
2. The second pass also involved the same treatment on a grid offset by 2.0 m from the original grid.
3. This was followed by an ironing pass involving general tamping of the ground with a reduced fall of 6 m for obtaining a level surface on the finished ground. The average total energy input applied considering all the passes was 2214 kN.m/m^2.

After dynamic compaction was done, the area was graded with ten passes of a 10 tonne vibratory roller. A minimum lag time of one week was given between each subsequent pass to allow the excess pore pressures to dissipate.

In-Situ Tests

After giving sufficient relaxation time to allow pore pressures to dissipate, a geotechnical investigation was done to assess the extent of improvement achieved. A comparison of SPT and q_c values before and after dynamic compaction is presented on Fig. 16.

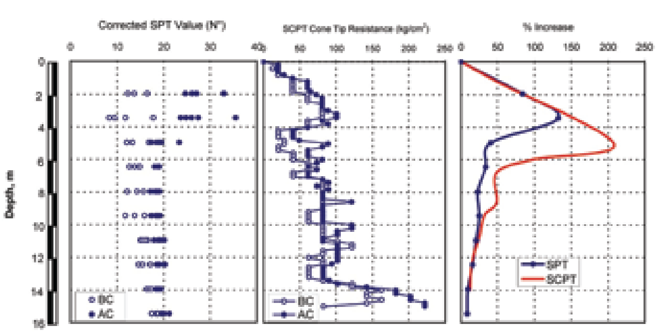


Fig. 16: SPT and SCPT before and after compaction

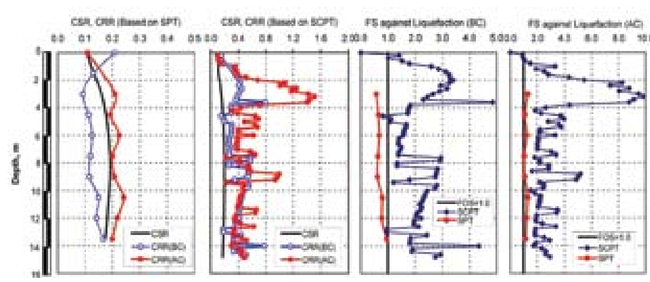
It can be seen that there is a 50 to 140% improvement in the corrected SPT values (N") to about 4 m depth, and 25 to 50% to about 11 m depth and 10 to 25% increase below to about 15 m depth. SPT values after compaction are generally greater than 16 to 20. The improvement in cone resistance values (q_c) varies widely from 50 to 210% in the top 4 m, 50 to 150% to about 10 m depth and 10 to 25% to 15 m depth. Cone resistance values after compaction are generally greater than 50 kg/cm^2 (5 MPa).

The peak improvement was observed between 1 m and 4-5 m depths. The treatment is effective up to a depth of about 9-10 m, below which the increase in SPT and cone resistance values is marginal.

Liquefaction Mitigation

Comparative plots of CSR, CRR and computed factor of safety against liquefaction (based on SPT and SCPT values) before and after compaction are presented on Figure 17.

It can be seen that the untreated site (before compaction) was susceptible to liquefaction to about 8 m depth. However, after compaction, the factor of safety against liquefaction (based on both SPT and SCPT values) is greater than 1.0. Thus, ground improvement by dynamic compaction has been successful in mitigating the liquefaction potential at the site.



* BC : Before Compaction AC : After Compaction

Fig. 17: Liquefaction susceptibility analysis before and after compaction

It can be seen that the untreated site (before compaction) was susceptible to liquefaction to about 13 m depth. However, after compaction, the factor of safety against liquefaction (based on both SPT and SCPT values) is greater than 1.0. Thus, ground improvement by dynamic compaction has been successful in mitigating the liquefaction potential at the site.

Foundation System

For the unimproved ground, pile foundations would have been necessary to transfer the structural loads below the liquefiable zone. As a result of ground improvement by dynamic compaction, open foundations bearing on the improved ground could now be provided. This resulted in substantial time and cost savings for the owner.

Isolated column footings with interconnecting plinth beams were provided for the Boys Hostel building. These foundations were designed for a net allowable bearing pressure of 175 kPa.

CONCLUDING REMARKS

The paper demonstrates the importance of in-situ tests to ensure effectiveness of ground improvement techniques to mitigate liquefaction potential and to ensure that the foundations are safe under the design loads. Field trials should be conducted to confirm the feasibility of the ground improvement method. For reliable and effective ground improvement process, sufficient in-situ testing before and after improvement is essential to ensure that adequate densification is achieved.

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