Stone Columns/Granular Piles for Improving Liquefiable Sites: Case studies

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ABSTRACT: Liquefaction is considered as a major hazard among different seismic risks. Ground improvement methods are commonly adopted to improve the liquefiable sites. The paper presents various aspects of liquefaction mitigation strategies to be implemented for liquefaction susceptible sites with focus on granular inclusions. A short discussion on liquefaction susceptible soils and its evaluation followed by outlines of the ground engineering applications is presented herein. Mechanisms that function at sites treated with stone columns/granular piles for liquefaction mitigation are discussed. Design aspects of granular piles for liquefaction mitigation are outlined. Few case studies, wherein stone columns have been adopted for improving the liquefiable sites, are presented. The paper concludes and highlights the effectiveness of granular inclusions in improving the liquefiable sites through various mechanisms.

KEYWORDS: Earthquakes, Liquefaction, Ground improvement, Stone columns, Granular piles

1. INTRODUCTION

Earthquakes are constantly posing risk to life and infrastructure facilities. Liquefaction, the state under which soil deposit loses its strength and flows as fluid is a major cause for damage during earthquakes. Liquefaction is manifested by the formation of sand boils and mud-spouts at the ground surface, by seepage of water through ground cracks or in some cases by the development of quicksand conditions over substantial areas (Seed and Idriss, 1982). Among the various seismic hazards, liquefaction is probably the most disastrous one leading to huge damage to structures and human life. Kutch earthquake of 1891 and the recent 2001 Bhuj/Kutch earthquakes have witnessed large areas that suffered damages due to liquefaction (Shankar, 2001).

Three approaches are available to reduce liquefaction hazards when designing and constructing new structures (JGS 1998). The first one is to avoid construction on liquefaction susceptible soils; the second one is to build liquefaction resistant structures and the third possibility is to strengthen the ground, by improving the strength, density, and drainage characteristics of soils, using a variety of soil improvement techniques. The third one is considered as the best strategy and is the most preferred choice. The basic strategy for the liquefaction mitigation is as shown in Figure 1 (PIANC 2001). Resistance to liquefaction can be improved by increasing the density, modifying the grain size distribution, stabilizing the soil, reducing the degree of saturation, dissipation of the excess pore pressures generated and intercepting the propagation of excess pore pressures, etc. Most common methods to achieve the above and to improve the engineering properties of the soils can be classified as densification, reinforcement, grouting/mixing and drainage.

Ground engineering techniques are commonly employed to mitigate liquefaction hazards. Various ground-engineering methods can be grouped based on the mechanisms to mitigate the liquefaction potential and damage. They are mainly: densification, solidification, replacement, drainage, lowering of ground water table, and shear strain restraint method. Among the various techniques, more widely used methods for liquefaction mitigation are: vibro methods (vibro-rod, vibro-compaction, vibro-replacement); deep dynamic compaction; compaction grouting; deep soil mixing; jet grouting; drainage, permeation grouting, explosive compaction and removal and replacement (Idriss and Boulanger, 2008). Yasuda *et al.* (1996) and Mitchell (2008) reported the application of various ground improvement methods to improve the liquefaction resistance of liquefiable sites and highlighted the effectiveness of granular inclusions, compared to other methods.

Granular inclusions in the form of stone columns/granular piles/drains, sand compaction piles have been extensively used for various ground improvement projects, which function in different mechanisms. This paper presents discussions on the application of granular inclusions in the form of stone columns/granular piles as a general ground improvement method as well as liquefaction hazard mitigation measure. Two case studies as described show the advantages of stone columns for improving the potentially liquefiable soils.

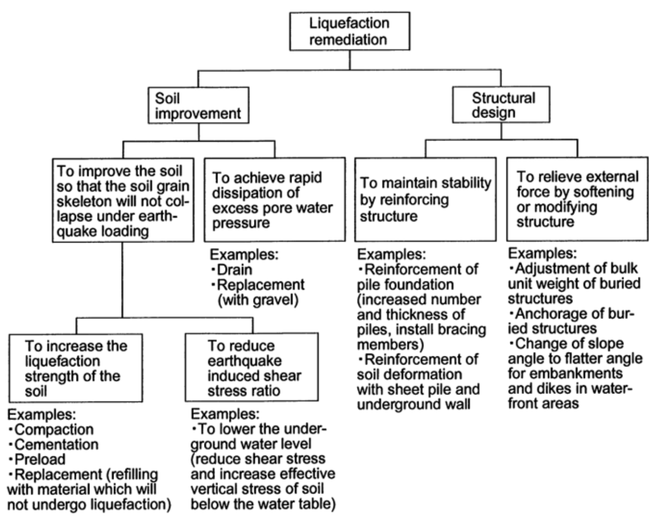

Figure 1 Strategy to be considered for liquefaction remediation (PIANC, 2001)

2. STONE COLUMNS/GRANULAR PILES

Granular inclusions in the form of granular drains/piles or stone columns are the most widely preferred, among the various ground engineering options, due to several advantages associated with them. One of the main benefits of ground treatment with granular piles is the densification of in situ ground by which the in-situ properties of the ground get modified to mitigate the seismic risks, especially the liquefaction potential. Densification by rammed granular pile (RGP) causes increase in deformation moduli and decrease in the coefficients of permeability and volume change. The densification effect decreases with distance from the centre of the compaction point and may become negligible at the periphery of unit cell (Krishna et al. 2006 & 2007, Krishna and Madhav 2009). Further, the very high deformation modulus and stiffness of the granular pile material provide reinforcement for the in-situ soil and together participate in resisting the earthquake induced shear stresses and lateral spreading. Granular piles function as drains and permit rapid dissipation of earthquake induced pore pressures by virtue of their high permeability. The generated pore water pressures, due to repeated loading, may get dissipated almost as fast as they are generated. In addition, they tend to dilate as they get sheared during an earthquake event. Seismic forces which tend to generate positive pore pressures in these deposits cause an opposite effect of dilation in dense granular piles.

Thus, different mechanisms operate in the functioning of stone columns/granular piles in liquefaction mitigation. These mechanisms are drainage, storage, dilation, densification and reinforcement. It was observed that granular piles are effective in mitigating liquefaction damage due to the reinforcement effect and drainage facility. Though granular piles are efficient in many ways in mitigating the seismic risks, liquefaction mitigation is the main advantage as it is associated with the drainage function, which is a very special feature as granular drains. An overview of the various mechanisms that function by granular piles in mitigating the liquefaction was presented by Madhav and Krishna (2008), Krishna (2011) and Krishna et al. (2014).

The diameter of the granular pile, spacing between the granular piles, pattern of the piles installation, information about the length of the pile (termination depth) and gradation details of the granular material etc. need to be defined in the design process. Barksdale and Bachus (1983) presented design aspects to be considered while designing the stone columns. IS 15284 and JGS 1998 are the other two good resources for granular piles design. Priebe (1995, 1998) demonstrated the design concepts for vibro replacement. In particular, Priebe (1998) proposed a relatively simple procedure to evaluate the improved liquefaction resistant capacity of vibro columns installed ground, based on stress transfer mechanism. The method did not consider the drainage mechanism of vibro columns. However, design curves developed by Seed and Booker (1977) are used in general for sizing the granular/gravel drains as a liquefaction remediation measure considering the drainage mechanism. To consider the densification effect in connection to liquefaction mitigation, improved SPT N values or cone tip resistance values could be correlated to cyclic resistance ratio and design accordingly (Baez, 1995).

Selection of gravel material for granular pile construction is very important, which governs the design and performance of the improved system. Stone columns generally use gravel or crushed stone as backfill. In general, coarse, open-graded stone of size varying from about 5 to 100 mm with not more than 15% material finer than 5 mm is being used for the construction of stone columns. However, the actual size of the granular material depends mainly on the gradation of in-situ soil. Saito et al. (1987) proposed a formula for the selection of the size of the material for stone column in relation to grain size of the surround in-situ soil as:

$$20D_s15 < D_G15 < 9D_s85$$

where D_s15 and D_G15 are the sizes/diameters (mm) of soil particles and gravel material, respectively, passing 15 percent and D_s85 is the diameter of soil passing 85 percent in a grain size analysis test.

Various techniques of installation have been conceived for various types of columnar inclusions in a wide variety of soils such as loose sandy to soft compressible soils depending on technical ability, efficiency and local conditions. Granular piles are installed by vibro-compaction, vibro-replacement, cased bore hole (rammed stone columns/RGP) or by simple auger boring methods (Datye and Nagaraju 1975, 1981; Balaam and Booker, 1981; Barksdale and Bachus, 1983; JGS, 1998). Figure 2 shows the common methods followed for construction of stone columns.

2.1 Stone Columns for Liquefaction Mitigation

In many instances, it is required to design the stone columns to prevent liquefaction. Design charts developed by Seed and Booker (1977) are being extensively used for the purpose (Figure 3). Figure 3 shows the variation of the greatest pore pressure ratio W_{max}, developed as a function of the spacing ratio, *a/b*, for values of N_{eq}/N_l equal to 2 and 4, and for a range of values of the dimensionless time factor $T_{ad} = (k/\gamma_w)(t_d/m_v a^2)$.

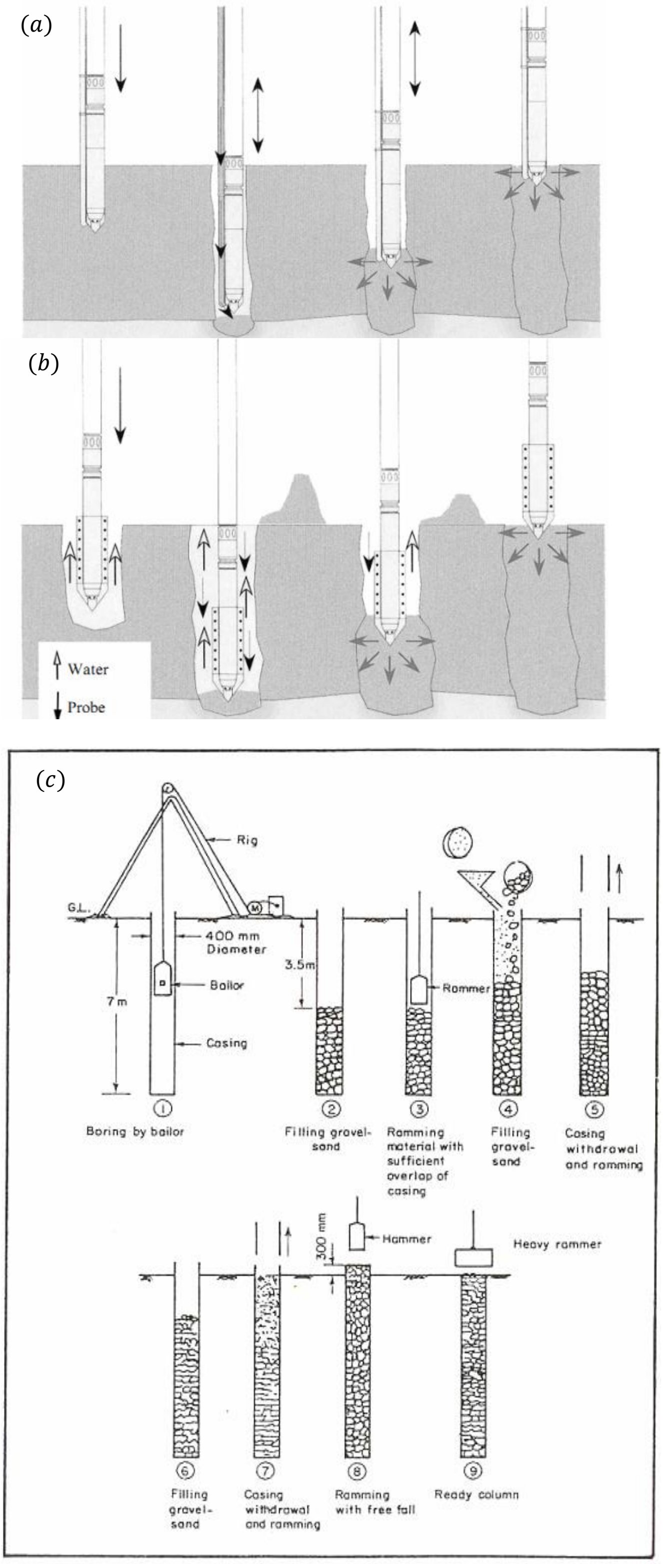

Figure 2 (a) Dry bottom-feed; (b) Wet top-feed method process (JGS, 1998), (c) Cased-borehole method (after Datye and Nagaraju,1975)

For any particular soil and a selected diameter of stone column, N_{eq}/N_l and T_{ad} will be known, and thus, the value of a/b corresponding to allowable value of W_{max} can be read directly from the curves. Design diagrams by Iai and Koizumi (1986) and Onoue (1988) incorporated the effects of drain resistance in the analyses of Seed and Booker (1977). Baez and Martin (1992) presented an evaluation of the relative effectiveness of stone columns for the mitigation of liquefaction of soil.

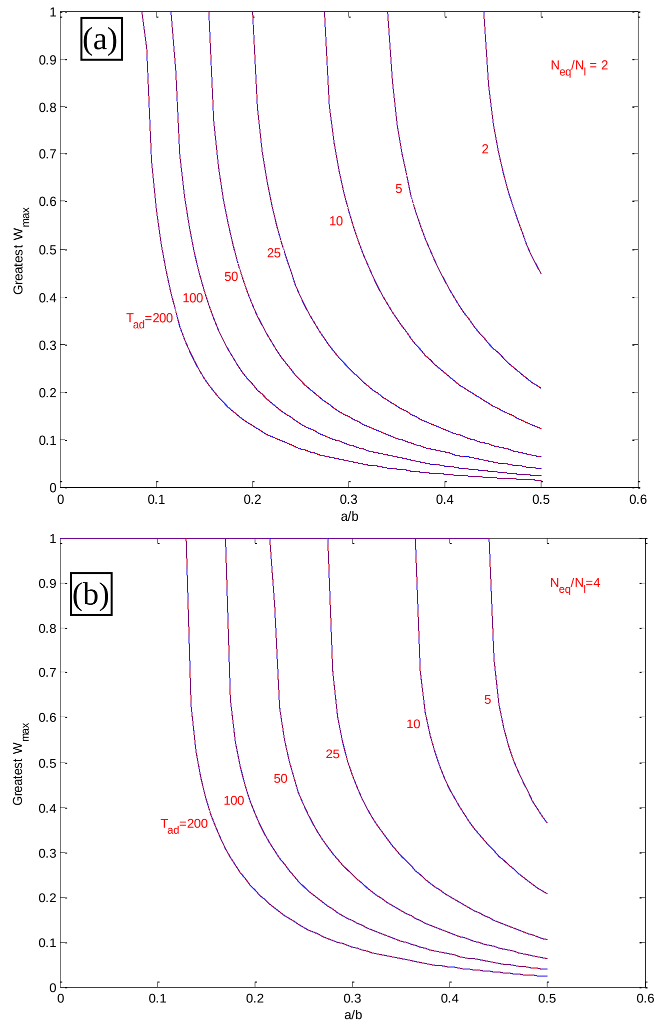

Figure 3 Relationship between greatest pore pressure ratio and drain system parameters for: (a) N_{eq}/N_l = 2, (b) N_{eq}/N_l = 4 (redrawn after Seed and Booker, 1977)

Pestana *et al.* (1998) analysed the development of excess pore pressure in a layered soil profile, accounting for vertical and horizontal drainage with a non-constant 'equivalent hydraulic conductivity', and head losses due to horizontal flow into the drain and the presence of a reservoir directly connected to the drain were considered. Boulanger *et al.* (1998) evaluated the drainage capacity of stone columns or gravel drains for mitigating liquefaction hazards. Dilation effect of the stone columns, due to densification around and within the stone columns, on the drainage function of granular piles was studied by Madhav and Arlekar (2000) by extending the Seed and Booker model (1977). It was shown that the dilation effect on pore pressure dissipation by granular piles for the range of parameters considered is marginal. Poorooshasb *et al.* (2000) demonstrated the effectiveness of inclusion of stone columns in reducing the risk of liquefaction of very loose to loose sandy and silty sand layers using the concept of equivalent permeability.

Poorooshasb *et al.* (2006) and Noorzad *et al.* (2007) demonstrated the reinforcement effect of stone columns while analysing their performance during an earthquake. They proposed that the seismic load imposed on the soil is shared between the stone column and the surrounding ground and stone column carries the major load. Krishna *et al.* (2006) studied the densification effect with respect to the coefficients of permeability and volume change at the near and at the farthest ends of the granular pile, individually and together, on maximum pore pressure variations during an earthquake event.

Krishna and Madhav (2008) combined both the densification and dilation effects and incorporated them in the analysis of pore pressure generation and dissipation. Bouckovalas *et al.* (2009, 2011) considered sand fabric evolution effects on drain design for liquefaction mitigation. They propose that overlooking the shake-down effects of fabric evolution during cyclic loading underestimates the effectiveness of gravel drains. Krishna et al. (2014) combined the soil fabric evolution effect and the densification effect in analyzing the pore pressures for the granular pile reinforced ground. Figure 4 shows the variation of the greatest pore pressure ratio W_{max} (peak values of W_{max} observed during the entire period of seismic loading, T_d), developed as a function of the spacing ratio, a/b, for values of N_{eq}/N_l equal to 3 and 4, and dimensionless time factor Tad = 5 to 200. The combined effect 'Densification and soil fabric effect', which is more general and realistic in terms of considering most possible effects, gives slightly lower values than that of the Seed and Booker (1977) curves. Krishna et al. (2014) provided a design example demonstrating the use of the design curves for liquefaction mitigation for a set of assumed parameters.

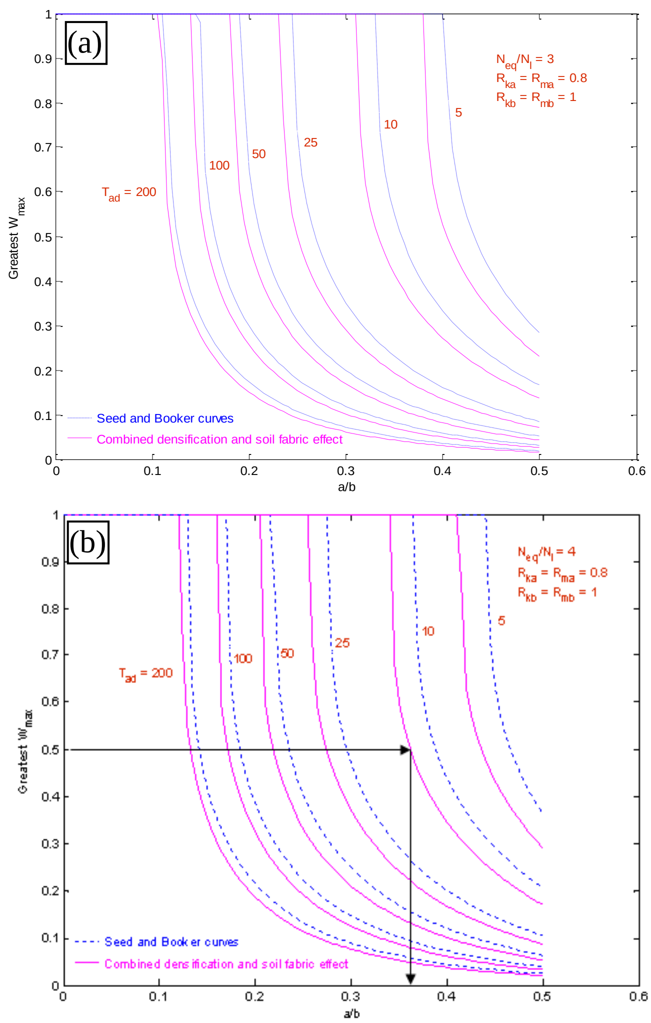

Figure 4 Design curves considering densification and soil fabric effect (a) N_{eq}/N_l of 3 (b) N_{eq}/N_l of 4, (after Krishna et al. 2014)

3. CASE STUDIES

Stone columns/granular piles have been adopted as ground engineering method for various projects for improving bearing capacity of the virgin ground and also as a liquefaction counter measure (Mitchell and Wentz, 1991). Adalier and Elgamal (2004) and Kumar and Raju (2012) summarized some of the field case histories on the use of stone columns as liquefaction counter measure. Case studies on implementation of stone columns for improving the liquefiable sites located in Northeastern region of India are presented in this section.

3.1 Case Study 1: Power Grid Project

At a Power Grid project site for HVDC station at Biswanath Chariali site located (Figure 5) in the Assam state (Zone V, according to IS 1893), the subsoil posed problems related to low bearing capacity and high liquefaction potential. Figure 6 shows the typical soil profile at the site along with SPT- N values variation with depth. The subsurface profile consists of top 2 m of poorly graded loose to medium fine sand followed by 4 m thick stiff silt underlain by medium dense Sand layer of 4 m. Further, firm to stiff Silt layer of 3 m was followed by dense Sand up to 15 m.

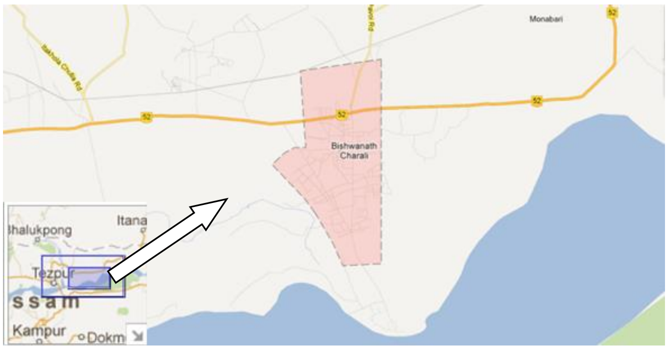

Figure 5 Location map of construction site

Liquefaction potential at the site has been evaluated as per the SPT N values based semi empirical method (Idriss and Boulanger 2008). An earthquake of 8.0 magnitude with peak ground acceleration value of 0.36g was considered for evaluating the liquefaction potential. Factor of safety (FOS) against liquefaction is presented in Figure 6 indicating liquefiable soil layers having FOS values less or equal to 1.0.

The occurrence of top and intermittent loose silty sand layers posed a challenge to the safe bearing capacity (varying from 120 kN/m^2 to 200 kN/m^2) and settlement (both total and differential) including liquefaction mitigation of the foundations for converter station structures. Ground improvement scheme using stone columns has been adopted for improving the bearing capacity and liquefaction mitigation. Wet vibro stone columns of diameter ranging 800 mm to 1100 mm for a length of 11.5 m were installed (KellerIndia 2015).

To confirm the spacing of the columns, initial load tests were conducted on 800 mm diameter stone columns installed at 2.4 m, 1.8 m and 1.6 m spacing. The load test results obtained for different spacing arrangements are shown in Figure 7. It is observed form the figure that 1.8 m spacing of stone columns gave optimum spacing, resulting in the similar maximum bearing pressure as that of 1.6 m spacing.

Further, liquefaction potential of the ground after the vibro stone column scheme has been evaluated using Pribe (1998) which indicated significant improvement in the liquefaction potential. The primary reason for this improvement is the stress reduction onto the soil due to the presence of vibro stone columns, which take significant portion of shear stresses. As per the Pribe (1998) method the stress reduction factor α was evaluated to be 0.47. Besides this stress reduction, existence of vibro stone columns facilitate drainage function and reduces the pore pressures that could be evaluated based on Seed and Booker (1977) method. For the *a*/*b* values of 0.33, 0.44 and 0.5 (for 0.8 m dia vibro stone column at 2.4, 1.8 and 1.6 m spacing), the pore pressure ratios were found to be well below 0.5 for wide range of T_{ad} values.

The particular project got the advantages of increased bearing capacity and reduced liquefaction potential due to installation of vibro stone columns of 0.8 m diameter at different spacing for different structures.

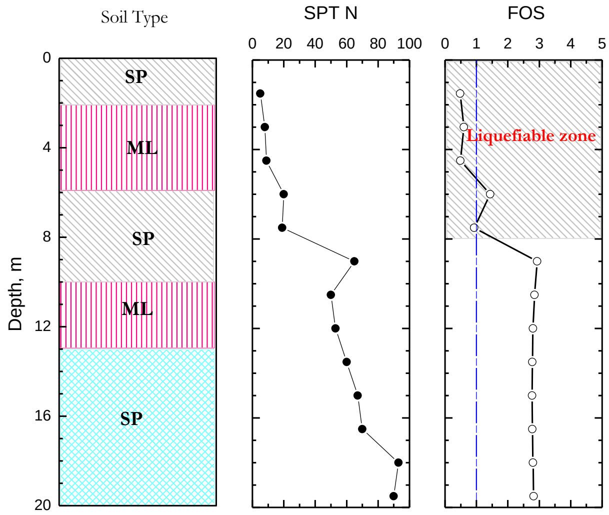

Figure 6 Typical soil profile and liquefaction potential evaluation

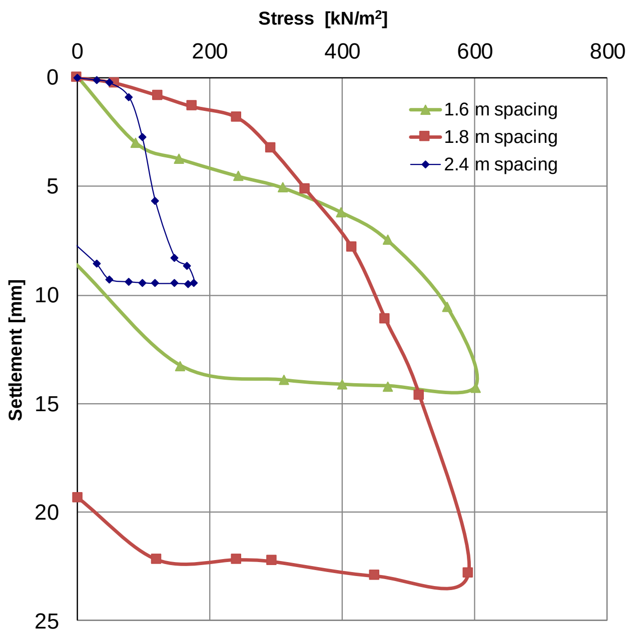

Figure 7 Load test results on stone column reinforced ground with difference configurations

3.2 Case Study 2: ROB project at Bongaigoan

As a part of National Highway expansion, National Highway Authority of India (NHAI) constructed two road over bridges (ROB) at Chaprakata, Bongaigaon in Assam. Reinforced Earth system is proposed to retain the ROB approaches having 10 m to 12 m high embankments near railway abutments. The ROBs consists of high embankment ramps, reinforced earth (RE) walls, abutments and other associated road infrastructure construction to facilitate the railway crossing.

Boreholes were explored to a depth of 7 to 15 m below the existing ground level. The subsurface profile revealed that top 4 m to 5 m of soil consists of soft to firm clayey/sandy silt and silty sand underlain by medium dense to dense silty sand with gravel up to borehole termination depth (about 15m) as shown in Figure 8. Ground water table was encountered at 1.0 m to 1.5 m below existing ground level.

The top soil layers being weak (SPT N~6), poses global instability and higher settlements for the ROB approach embankments and abutments. In addition, as Bongaigaon falls in seismic zone V as per (IS 1893), liquefaction related issues and global stability with respect to static and seismic conditions of the retaining wall was another major concern. Figure 9 shows the typical factor of safety values (as per Idriss and Boulanger 2008) for one borehole data. Keeping the above geotechnical challenges for the project, ground improvement scheme with vibro stone columns was adopted for the project to support the high embankment.

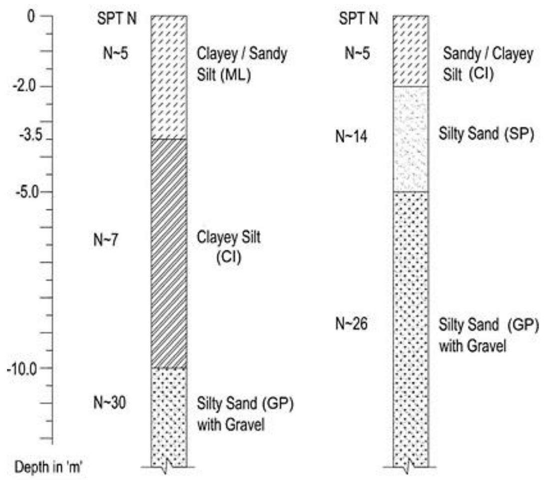

Figure 8 Typical soil profiles at the ROB sites in Bongaigaon

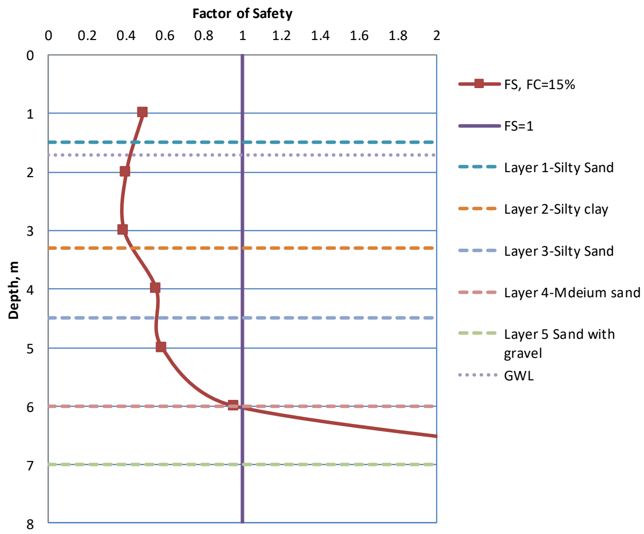

Figure 9 Typical liquefaction potential analysis (factor of safety variation) at ROB site

The design analysis of vibro stone columns as carried out according to Priebe's (1995) design methodology and finalized the design of 0.9m diameter vibro stone columns at 2.2 to 2.5 m spacing in square grid and. The depth of treatment was adopted as 4 m to 10 m based on the soil profile and height of the RE Wall embankment. The properties of stone column material considered were: Unit weight: 19 kN/m^3 (submerged: 12 kN/m^3); Angle of internal friction: 42^o; Modulus of compressibility: 120,000 kPa. Typical ground improvement scheme adopted is shown in Figure 10. In order to avoid difficulties prevailing at the site, such as non-availability of water nearby for installation of stone columns, muck generation and muck removal for wet method, installation of vibro stone columns using dry bottom feed method has been adopted.

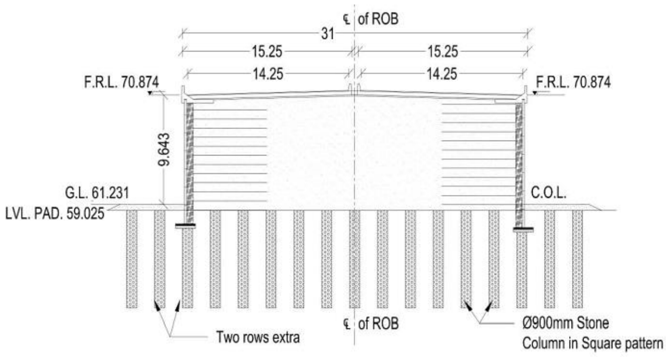

Figure 10 Typical ground improvement scheme adopted for ROB site in Bongaoigaon

Analyses with the designed ground improvement scheme using vibro stone columns indicated enhanced bearing capacity and reduced liquefaction potential as which ensured the safety of the structure. It was also found that differential settlements were significantly reduced with ground improvement scheme as vibro stone columns converted the variable fill layer to a homogenous mass. The technique provides a cost effective solution for treating the combination of soft cohesive and loose non-cohesive deposits. Besides improving the shear strength and compressibility parameters of the in-situ soil, the technique also provides effective drainage paths to ensure rapid consolidation.

4. CONCLUSIONS

Liquefaction is the most disastrous feature during an earthquake that causes huge loss and damage to various structures built on or in the ground. Ground improvement methods are extensively used to enhance the *in situ* ground performance for liquefaction mitigation. Among the various options available the most widely used method for mitigating the liquefaction hazard is the installation of stone columns/granular piles/drains. Mechanisms such as reinforcement, densification, drainage along with storage and dilation mitigate the damages due to liquefaction. The design of granular piles for liquefaction mitigation is commonly done using the design charts developed based on analytical model representing pore pressure generation and dissipation considering various effects. The paper presented short summary of recent developments in the design of stone columns for liquefaction mitigation considering drainage. Further, two case studies were presented to demonstrate the effectiveness of vibro stone columns for improving bearing capacity and mitigate the liquefaction potential.

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5. NOTATION

a radius of the granular pile
b radius of the unit cell
$k_h(r)$ horizontal permeability of treated ground at radial distance r
k_{hi} horizontal permeability of untreated ground

$m_v(r)$ coefficient of volume compressibility or treated ground at radial distance r
m_{vi} coefficient of volume compressibility or untreated ground
N equivalent number of uniform stress cycles associated with any period of earthquake shaking
N_{eq} equivalent number of uniform stress cycles induced by earthquake
N_l number of uniform stress cycles required to cause liquefaction
N_{eq}/N_l cyclic ratio
R non-dimensionalized radial distance, r/b
r radial distance measured from the centre of granular pile
T normalized time, t/t_d
t time
$T_{ad} = (k/\gamma_w)[t_d/(m_v a^2)]$ dimensionless time factor
$T_{bd} = (k/\gamma_w)[t_d/(m_v b^2)]$ dimensionless time factor
T_s period of the earthquake shaking
t_d duration of earthquake
u excess hydrostatic pressure
u_g excess hydrostatic pressure generated by earthquake shaking
W or r_u pore pressure ratio
$W_{\max}$ maximum pore pressure ratio W throughout the layer at a given T
σ'_o the initial mean bulk effective stress
α a non-dimensional parameter describing the pore water pressure generation during shaking

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