Effects of the Length of Jet Grouted Columns and Soil Profile on the Settlement of Shallow Foundations

ZAHEER AHMED ALMANI* ASHFAQUE AHMED MEMON*, AND NAEEM AZIZ MEMON**

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ABSTRACT

In this paper, the effect of length of jet grouted columns and varying soil profile under shallow foundations of buildings constructed on the liquefiable ground was studied. The isolated shallow footing pad which supports a typical simple frame structure was constructed on the liquefiable ground. This ground was reinforced with jet grouted column rows under the shallow foundations of structure. The system was modeled as plane-strain using the FLAC 2D (Fast Lagrangian Analysis of Continua) dynamic modelling and analysis code. This case focuses on the length of jet grouted columns in a soil profile and the effect of soil profiles of varying thickness on the settlements of building structure when the soil is liquefied during an earthquake. The results show that liquefaction-induced large settlements of shallow foundation of building decrease to tolerable limits with the increase in the length of columns. For soil profiles, with a relatively thinner liquefiable layer, a certain minimum length of columns (extended in base non liquefiable layer) is required to meet the settlement tolerable limits. For soil profiles, with a relatively thicker liquefiable layer, this length should be equal to the thickness of the liquefiable layer from the footing base plus some extension in the base non liquefiable dense layer. In the soil profile with the base liquefiable layer underlying the non liquefiable layer, settlements could not be reduced to the tolerable limits even with columns of relatively larger length which may be critical.

Key Words: **Liquefiable Ground, Jet Grouted Columns, Length of Columns, Soil Profile, Numerical Modelling.**

1. INTRODUCTION

Karachi biggest commercial hub of Pakistan is lying in the region most vulnerable to earthquakes which usually originate from epicenter at the Gujrat fault. In coastal areas, soil deposits may be susceptible to liquefaction in the event of earthquake. The buildings in this area may likely be damaged due to liquefaction-related large settlements.

The buildings on shallow foundations constructed over liquefiable loose or medium dense sand deposits where water is high, suffer significant damage due to liquefaction-induced settlements and tilting [1,2]. These large settlements as large as 1 m are initiated, due to, bearing capacity or shear failure when the soil looses stiffness as a result of liquefaction and also due to reconsolidation

* Assistant Professor, and ** Lecturer,
Department of Civil Engineering, Mehran University of Engineering & Technology Jamshoro.

(densification). This reconsolidation of soil occurs as pore water is dissipated causing volume change of the soil. This type of response of shallow foundations have been reported during earthquakes such as Kishida, [1], Ohsaki, [2], Seed, et. al. [3] and Martin, et. al. [4-5].

The ground reinforcement with stiff high modulus jet grouted/deep mixing column rows, requiring relatively small replacement area in the ground, which has recently demonstrated its performance in earthquakes such as Kocaeli, Turkey, and Kobe, Japan, [5-6] may be cost effective and easy to install. Further, with relatively less replacement area, transmission of motion towards structures may be relatively less.

Further, ground reinforcement using vertical stone columns involves larger settlements which may be intolerable for structures [7]. This requires that relatively stiffer treatment with cemented columns be adopted for structures to meet the tolerable settlement limits [7]. In addition, stone columns installation may cause vibration-induced settlements. Further, little work has been focused on the optimum geometry of deep mixing/jet grouted columns in the ground to limit the settlements of shallow foundations to meet the tolerable limits of the existing buildings. In this regard, the effects of area, depth and position of treatment relative to building structure on the performance are particularly important. Numerical modeling needs to be carried out to study the effects of number and length of jet grouted columns in liquefiable soil layer under the shallow foundations of structure.

Thus, in order to get optimum treatment, this study is the part of comprehensive research carried out on jet grouted columns applied as liquefaction remediation technique to mitigate the damages to the foundation of structure during earthquakes at Nottingham Centre for Geomechanics.

Therefore, in this study, the effect of length of jet grouted columns on treatment performance in the soil profile was analyzed. Moreover, the effect of soil profiles with varying thickness on the treatment performance is also important because the construction site at a structure may have varying soil profiles. Further, in this study various soil profiles were taken to study the effect of those profiles on the treatment performance.

The objectives of this research were to:

(1) Investigate the effect of length of jet grouted columns on the treatment performance.

(2) Investigate the effect of varying thickness of soil profile on the treatment performance.

(3) Investigate the effect of varying soil profiles on the treatment performance.

2. THE CASE TAKEN FOR ANALYSIS

One typical case of building on shallow foundation, founded on the natural ground with design parameters as shown in Fig. 1 was evaluated.

2.1 The Structure and the Soil Deposit

For this study, the isolated footing of 4x4m pad size and 1m thickness was constructed at 1m depth from the ground surface. This footing supports a central column of 1x0.5m cross-section and 5m in length, which supports a part of superstructure, as shown in Fig. 1.

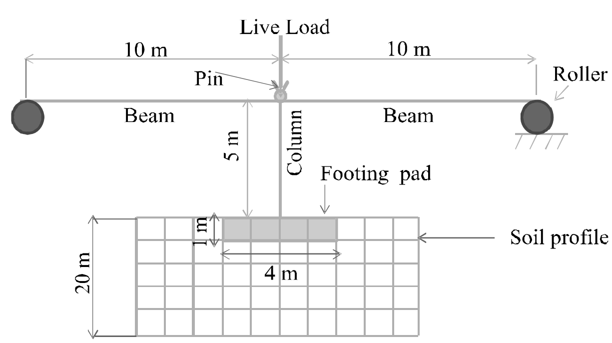

FIG. 1. SUPERSTRUCTURE AND FOOTING OF 4m WIDE x1m DEEP (MESH AND STRUCTURE NOT DRAWN AS PER SCALE BUT SHOWS MODEL QUALITATIVELY)

The simplified superstructure considered for this study consists of four rectangular beams, each with cross-sectional dimensions of 1x0.5m and a length of 10m. The beams are joined together and are pin-supported on the central column at a right angle. The far ends of all the beams are assumed to be simply supported over the roller supports. The column and beams are modelled by structural beam elements provided with structural logic in FLAC. The concrete footing pad is represented by grid elements which are represented by the liner elastic constitutive model with elastic properties (shear modulus, bulk modulus) and density of structural concrete. The structural elements have rigid connection with the footing grid, representing the column to be rigidly attached to the footing pad. The beams and columns were assigned masses so that the dead load is properly transferred to the column and then to the footing pad before shaking. During shaking, the superstructure exerts inertial horizontal force on the column and footing pad. As the superstructure beams were supported at their far ends with rollers and the live load was applied at (the pin jointed) top of column, one half of the total live and dead load is transferred to the top of the column and onto the pad. The allowable footing bearing pressure for the medium dense sand taken as the foundation soil is in the range of 100-30kPa as per CP 2004, 1972. The lowest footing allowable bearing pressure of 100kPa in this range for the foundation soil (medium dense sand) was considered in this specific case. The magnitude of the live load on the top of the column was considered in such a way that the bearing pressure on the soil due to both the dead and the live load is 100kPa. In FLAC 2D plane strain modelling, the structural elements (beams and columns) are assumed to be extended continuously in the transverse direction when in reality they are installed at certain spacing. Therefore, the Young's modulus and the density were divided by the spacing for FLAC 2D input.

The soil profile with two layers shown in Fig. 2 was taken for this study. This soil deposit consists of liquefiable medium dense (LB) E-Fraction Leighton Buzzard silty sand (being a typical liquefiable sand) layer with thickness of 10m as the surface layer (at 40% relative density). Underlying this surface layer, non liquefiable dense layer (at relative density of 80%) with thickness of 10m was provided. Further details relating to structure, its properties and soil profile have been described in detail in Almani, et. al. [8].

2.2 Numerical Modelling Code Selection and Coupling of Modules

The liquefaction and its mitigation can be modelled and analyzed by numerous methods. The degree of coupling in these codes range from full to partial coupling.

Finite Difference Method with FLAC 2D computer code Version 6.0 was chosen for numerical analysis. Liquefaction problem was modelled in this partially-coupled solution code by coupling the dynamic module with ground water flow module. For more details see the FLAC User's Manuals [9].

2.3 Earthquake Excitation

For obtaining consistent results, the sinusoidal horizontal velocity wave (lateral loading wave) with the amplitude of 0.2 m/sec was applied at the bottom of model. This design earthquake simulates a heavy earthquake with a

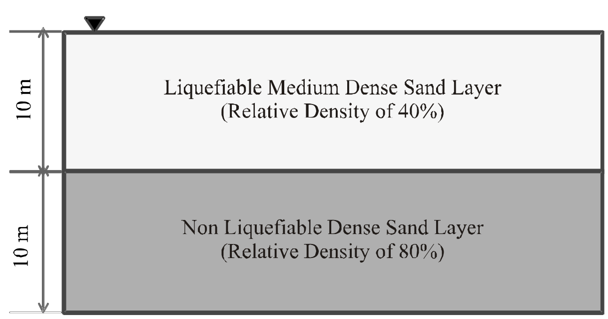

FIG. 2. THE SOIL PROFILE TAKEN FOR ANALYSIS

horizontal component of ground acceleration of 0.35g (app. 3.5m/sec^2) for duration of 10 sec. The simulation was also continued for a few seconds after the wave stops at 10 sec to bring the model in static equilibrium condition.

2.4 Basic Soil Properties, Constitutive Model and Dynamic Damping

The Finn/Byrne model in FLAC 2D for modelling the phenomenon of liquefaction is formulated on Mohr-Coulomb elastic-perfectly plastic failure criteria in conjunction with hysteric damping model as shown in Equation (1):

$$\Delta\varepsilon_v = C_1 \exp\left(\frac{C_2 \varepsilon_v}{\gamma}\right) \quad (1)$$

In Equation (1), $\Delta\varepsilon_v$ is volumetric strain increment in each cycle, ε_v is the accumulated volumetric strain from previous cycle and γ is the shear strain for the cycle, C_1 and C_2 are constants. The Hardin-Drnevich hysteretic damping strain constant γ_{ref} for the Hardin-Drnevich hysteretic dynamic damping model was taken as 0.05 for sand, as recommended in the FLAC 2D manual [9].

This combined constitutive model presents the liquefaction behaviour of soil [9] which is used for this analysis. With this model, decrease of shear modulus and increase of damping ratio with strain level during dynamic simulation occur as per modulus reduction graph given by Seed, et. al. [10].

The elastic shear and bulk modulus were calculated using the Hardin-Drnevich Equations (2-3) are given for sandy soils [11].

$$G_{max} = 100\left(\frac{3-e}{1-e}\right)^2 (\rho)^{0.5} \quad (2)$$

Where e is the initial void ratio and p' is the confining pressure.

$$B = \frac{2G_{max}(1-\nu)}{3((1-2\nu)} \quad (3)$$

Where v is the Poisson's ratio of soil.

The elastic shear and bulk modulus of the layers were taken as varying from ground surface because it is the function of confining stress.

The plastic property like drained peak friction angle was measured using consolidated drained triaxial test [12]. The Constant head permeability test was used to measure the permeability of soil. The relationship between shear and volumetric strains is defined by Finn/Byrne model with soil model parameters C_1 and C_2; therefore, the dilation angle which also expresses the same characteristics was taken as zero. The soil properties are given in Table 1.

For determination of model parameters, the Finn/Byrne soil model was calibrated by simulating the constant volume cyclic simple shear tests in FLAC 2D as single element test as described in detail in Almani, et al. [8]. Then trial and error method was applied to get the best values of the soil model parameters C_1 and C_2 and the

TABLE 1. PROPERTIES OF SOIL LAYERS

Property	Soil Layers	
	Medium Dense Layer	Dense Layer
Relative Density	40%	80%
Unit Weight (KN/m^3)	18.80	19.57
Porosity (Void Ratio)	0.47 (0.88)	0.42 (0.72)
Permeability (m/sec)	2x10^{-7}	1x10^{-7}
Peak Friction Angle (Degree)	32	48
Pore Pressure Constants	C_1=1.2; C_2=0.33	C_1=0.43; C_2=3.75
Hardin-Drnevich Damping Constant (γ_{ref})	0.05	0.05
Water Bulk modulus (kPa)	5x10^5	5x10^5
Water Tension (kN/m)	1x10^2	1x10^2
Water Density (kg/m^3)	1000	1000

Hardin-Drnevich hysteretic damping strain constant (γ_{ref}) shown in Table 2. These values were used as model parameters in the Finn/Byrne equation [1].

2.5 Ground Reinforcement

The ground was reinforced with stiff jet grouted/deep mixing circular column rows of a relatively same small diameter of 0.6m (or 0.5x0.5m square columns with cross-sectional area equal to circular columns) in all studies. The column jet grouted material (cemented sand) was represented with the Mohr-Coulomb soil model combined with Hardin-Drnevich hysteretic dynamic damping model during dynamic analysis (to incorporate the reduction of shear modulus and increase of damping ratio with strain level). The variation pattern of shear modulus and damping ratio was taken same in both sands because it in narrow band as found in cyclic tests [13]. The input model parameters such as Elastic, Shear, and Bulk modulus; plastic properties such as cohesion and tensile strength were determined in the laboratory as shown in Table 3. As columns are located below the water table, they were saturated like the soil.

TABLE 2. MODEL PARAMETERS

Model Parameters	Medium Dense Sand Layer	Dense Sand Layer
C_1	1.2	0.43
C_2	0.33	3.75
γ_{ref}	0.05	0.05

TABLE 3. PROPERTIES OF JET GROUTED COLUMN MATERIALS

Properties	Values
Saturated Unit Weight (KN/m^3)	19
Shear Modulus, G(kPa)	2×10^6
Bulk Modulus, K (kPa)	$2.6\text{x}0^6$
Unconfined Compressive Strength at 28 Days Curing Time (kPa)	4800
Friction Angle (Degree)	0
Cohesion (kPa)	2400
Water Cement Ratio	`1:1
Tensile Strength (kPa)	480
Permeability (m/sec)	1×10^{-8}

2.6 Tolerable Movement Criteria of the Building Structure

The settlements tolerable to the building structures in the event of earthquake given in building codes depend upon the type of building, nature of its components, functional use and dimensions. In the view of this, settlement criteria with one specified limits is not recommended in building codes. For a typical case of building in this study, settlements for the structure were quantified in tolerable limits based on the recommendations given in literature.

(1) Tolerable limits of 4cm recommended by Skempton and MacDonald [14]. These are the design limits for maximum settlements up to which building is in serviceable condition.

(2) Tolerable limits of 5cm recommended by European Committee for Standardization [15]. These are also the design limits for maximum settlements up to which the building is in service condition.

(3) Limits of 10cm at which the structure loss serviceability but may not collapse [16].

3. Model Development

For this study, the rectangular mesh with uniform zone size of 0.5x0.5m and the aspect ratio of one (1) was considered as shown in Fig. 3.

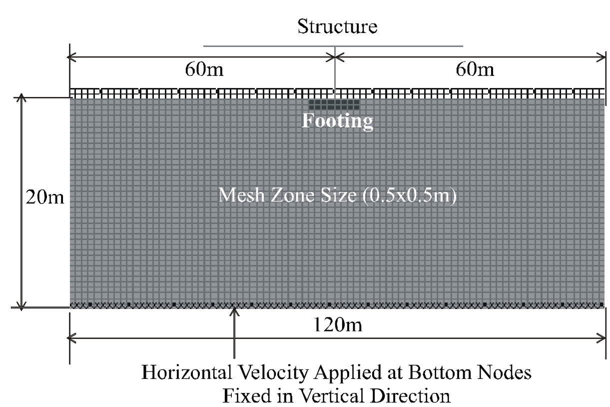

FIG. 3. FINITE DIFFERENCE MESH USED FOR FLAC 2D NUMERICAL ANALYSIS

For the static analysis, fixed lateral and bottom boundaries were taken while for dynamic analysis (earthquake loading), free field boundaries lateral boundaries of the model were taken. The lateral boundaries were placed at such a distance that the behaviour of the soil-structure system (stresses, strains and pore pressure) in the area of study remains unaffected due to boundary effects. For this analysis, the model with boundaries 3 times the deposit thickness or 60m each side of centre of footing was selected.

In order to depict hydraulic boundaries, the pore pressures with the top boundary of model were locked at zero values to make a free drainage top surface, whereas lateral and bottom boundaries were made impervious. More details regarding model development are described in Almani, et. al. [8].

3.1 Validation of the Numerical Model with Centrifuge Model

The numerical simulations at prototype scale as shown in were performed of the 2D plane strain centrifuge model test as shown in Fig. 4, under a centrifuge acceleration of 50g, carried out at Cambridge geotechnical laboratory. In this physical 2D plane strain test, a simple structural frame with spread footing of 4m width with bearing pressure of 100kPa was founded on the 18m full depth liquefiable medium dense layer at the relative density of 50%. The monitoring points are labelled in Fig. 5. The dynamic excitation in the form of a sinusoidal acceleration wave of 0.2 m/sec^2 with a frequency of 1.0Hz. was applied for the duration of 10 seconds.

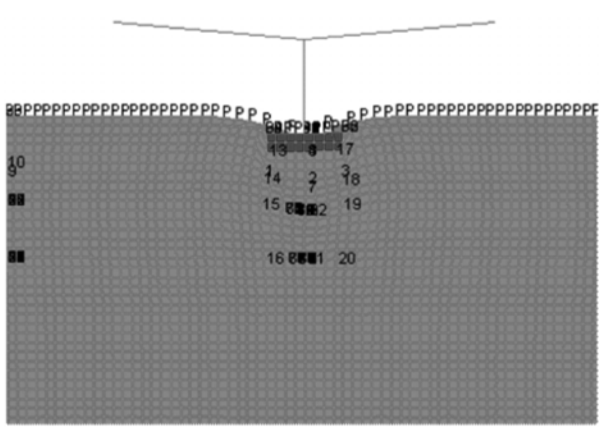

FIG 4. NUMERICAL FLAC 2D PLANE STRAIN MODEL

The settlements of footing as shown in Figs. 6-7 which is the variable of primary interest in this study, showed the same behaviour in both types of models in both the untreated and treated cases. This approach of numerical modelling with the constitutive soil model is in good agreement with the physical tests and therefore has been adopted for this study.

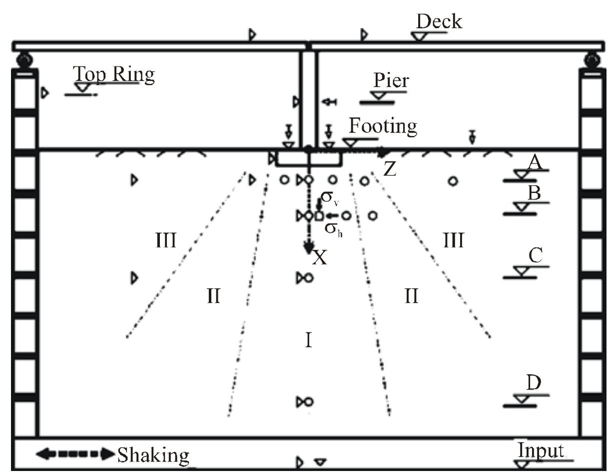

FIG. 5. CENTRIFUGE PHSICAL PLANE STRAIN MODEL

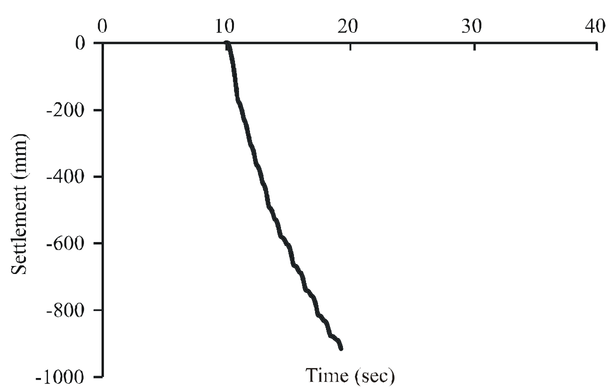

FIG. 6. SETTLEMENT OF FOOTING VERSUS TIME-FOR THE FLAC 2D MODEL

4. RESULTS AND DISCUSSION

In this study, the isolated shallow strip footing of the structure was founded on the ground reinforced with rows of grouted columns around the footing pad in three treatment geometries A, B and C. In A or adjacent type geometry treatment is provided adjacent to the footing pad. In B type geometry, treatment is provided beneath the footing pad and in C (combined) type geometry, treatment is provided adjacent and beneath the footing pad.

4.1 Effect of Length of Columns

In order to study the effect of length of columns on the treatment performance, the length of columns was varied in this the specific soil profile for all the three treatment geometries, Adjacent (A), Beneath (B) and Combined (C) as shown Fig. 8.

For the adjacent or A type geometry (rows of columns are provided adjacent to footing pad), the results show that the settlements exceed the limits of 10cm when the length of columns is up to 6m from footing base. The settlements come within the limits of 10cm with 7m length of columns because relatively less vertical deformations of liquefied soil occurs under the tips of columns due to the support provided by base non-liquefiable dense layer near the tips of columns. The settlements decrease to the tolerable limits of 5cm with 11m length of columns when columns are extended by 2m in base non liquefiable layer which remain in the same limits with further extension of columns up to 16m. For columns as long as 17m from footing base, the settlements further decrease to the tolerable limits of 4cm.

For beneath or B type geometry, the results show that the settlements decrease to the limits of 10cm with 8m long columns from footing base when their tips are near the base dense layer and the soil under the tip deforms relatively less in a vertical direction due to support from the base dense layer. When the length of the columns is 15m from the footing base, the settlements further decrease to the tolerable limits of 5cm.

For combined or C type geometry, the results show that the settlements decrease to the limits of 10cm when the length of the columns is 7m from the footing base. When the length of the columns is 11m so that they are extended by 2m in the base non liquefiable dense layer, the settlements further decrease to the tolerable limits of 5cm.

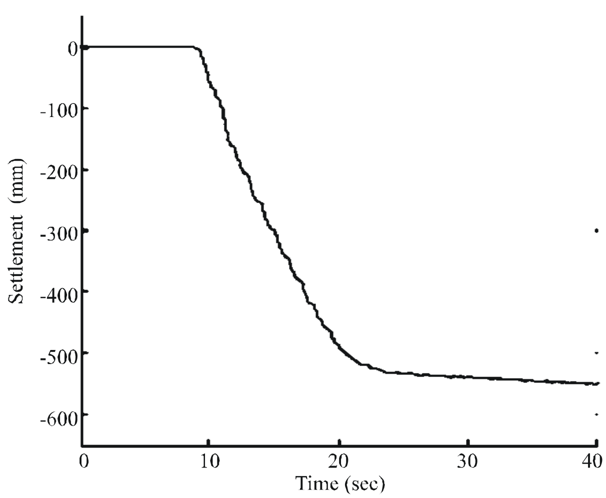

FIG. 7. SETTLEMENT OF FOOTING VERSUS TIME-CENTRIFUGE MODEL

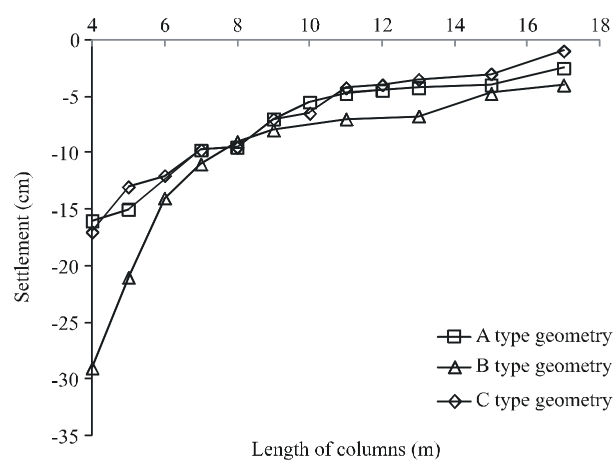

FIG. 8. SETTLEMENT VS. LENGTH OF JET GROUTED COLUMNS

The reason for the large settlements for the case when the columns are as short as 4m may be explained by drawing the contours of vertical displacement as shown in Fig. 9. The contours show that shorter columns with their tips in surface liquefiable layer, punch with larger displacement at their tips in that layer, when that surface layer liquefies, due to which the footing pad displaces in the vertical direction.

The results suggest that the optimum length of the columns should be 11m from footing base (depth of liquefiable layer under footing base plus 2m extension in base non liquefiable layer) to reduce the settlements to the tolerable limits, except for B type geometry for which longer columns of 15m are required. For the length of columns 7-8m (80-90% of the thickness of surface liquefiable layer) from footing base, the settlements come within the limits of 10cm. The results reveal that for improvement of the settlements to the tolerable limits, just resting on or embedding the columns in the base dense non liquefiable layer, is not sufficient but a certain length of the columns is required so that the tips of columns in base non liquefiable dense layer are embedded by around 2m. At this tip level of columns the soil has sufficient stiffness and bearing capacity to resist the shear punching of the columns. This is the depth at which liquefaction front has not reached and not reduced the stiffness and strength of soil.

4.2 Effect of Soil Profile

In the first series of models, effect of surface liquefiable soil layer of varying thickness on the treatment performance was studied by varying thickness of surface liquefiable layer, in the two layer soil profile as shown in Fig. 10 Error! Reference source not found.

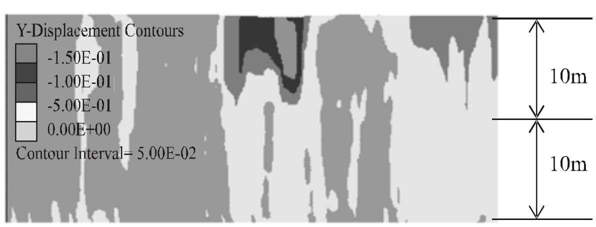

FIG. 9. VERTICAL DISPLACEMENT CONTOURS IN THE CASE FOR SHORT COLUMNS OF 4m LENGTH

The results for A and B type geometries as presented in Fig. 10 Error! Reference source not found, show that the settlements are in the limits of 5 and 10cm respectively when the thickness of surface liquefiable layer is 10m or less. In this case, thickness of surface liquefiable layer is relatively thin so that the columns (11m long from footing base) are extended by at least 2m inside the base non liquefiable dense layer. The settlements are within limits of 10cm when the thickness of liquefiable layer is such that the tips of the columns are resting on the top of base non liquefiable dense layer. The settlements generally increase as the thickness of liquefiable layer increases, though it remains in the limits of 10cm (slightly larger with B type geometry) in the case when thickness of liquefiable layer is such that tip of columns are near to the top of base dense layer. In this case non liquefiable base dense layer restrains the vertical deformations of liquefied soil under the tips of columns.

The results suggest that the settlements remain in the tolerable limits as long as the thickness of that surface liquefiable layer is so thin that columns are extended by at least two meters in base dense layer. The settlements increase and exceed the tolerable limits when the thickness of liquefiable layer is so thick that the tips of columns are not extended inside the non liquefiable stiff base layer.

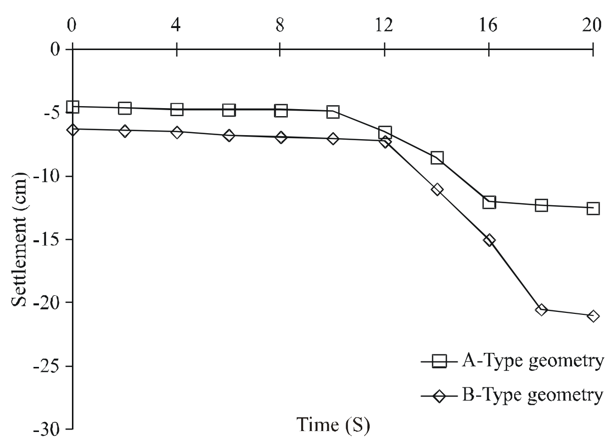

FIG. 10. SETTLEMENT OF FOOTING VERSUS THICKNESS OF SURFACE LIQUEFIABLE LAYER

In second series of models, the effects of the surface non liquefiable dense layer of varying thickness as the foundation soil and an underlying liquefiable base layer of varying thickness in two layer soil profile, on treatment performance was studied as shown in Fig. 11.

The results for A and B geometries as presented in Fig. 12, show that the settlements exceed the limits of 10cm when the thickness of surface non liquefiable dense layer overlying the base liquefiable layer is 12m or less. In this case the tips of columns are either lying inside or on the top (surface) of underlying liquefiable base layer. The settlement decrease to the limits of 10cm when the thickness of surface non liquefiable dense layer is as large as 14 and 16m for A and B type geometry respectively. In this case, tips of columns embed inside the non liquefiable layer by 2 and 4m for A and B type geometries respectively. The settlements remain in the same limits with further increase in thickness of surface non liquefiable dense layer except for the case of full depth non liquefiable dense layer when the settlements reduce to tolerable limit of 5 cm with A type geometry.

These results suggest that presence of surface non liquefiable dense layer as foundation layer of relatively smaller thickness has no effect on the reduction of the settlements unless the thickness of that surface layer is large enough that tips of columns embed inside that dense layer to the depth where they can get adequate support in the case when the underlying layer is liquefiable in the soil profile. Further, due to presence of underlying liquefiable in the soil profile settlements could not be reduced to the tolerable limits.

In the third series of models, the effect of the presence of the liquefiable layer in the centre of the surface and base non liquefiable dense layers on the treatment performance was studied. The thickness of this liquefiable layer was taken from 2-12m in the centre of three layer soil profile as shown in Fig. 13.

The results for A type geometry, as presented in Error! Reference source not found, show that when the full depth of the soil profile is non liquefiable dense layer, the settlements are within the tolerable limits of 5cm. Due to

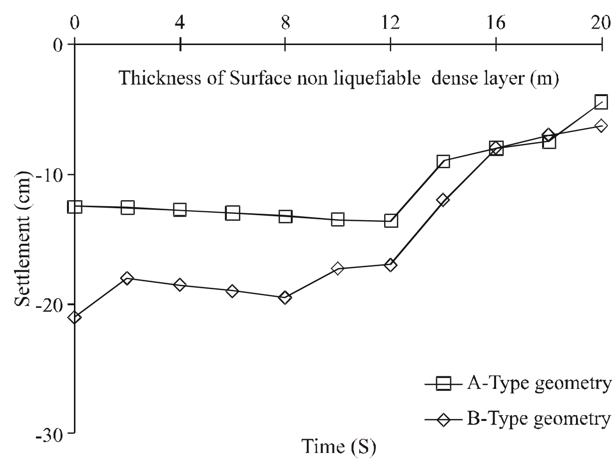

FIG. 12. SETTLEMENT OF FOOTING VERSUS THICKNESS OF SURFACE NON LIQUEFIABLE LAYER

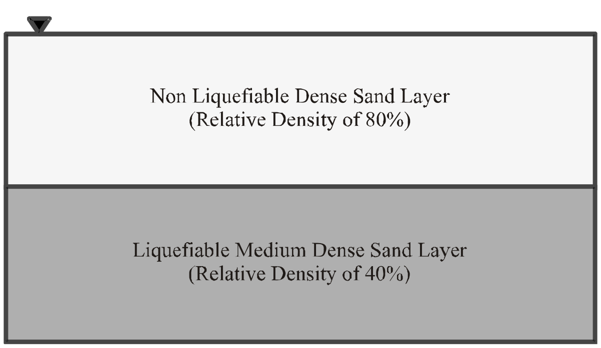

FIG. 11. TWO LAYER SOIL PROFILE WITH SURFACE NON LIQUEFIABLE LAYER

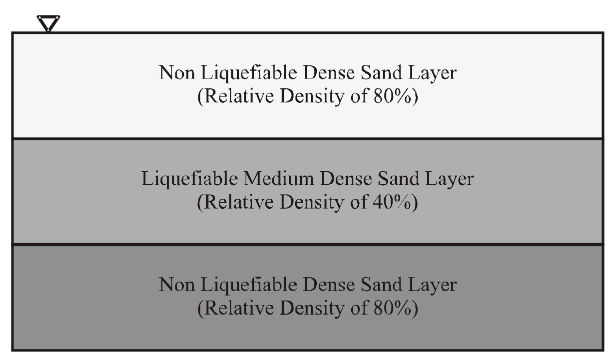

FIG. 13. THREE LAYER SOIL PROFILE WITH CENTRE LIQUEFIABLE LAYER

the presence of the liquefiable layer of thickness 2-8m in the centre of the soil profile, the settlements increase to the limits of 10cm. The settlements exceed the limits of 10cm when the thickness of the centre liquefiable layer is as large as 12m.

The results for B type geometry presented in Fig. 14 show that when the full depth of the soil profile is non liquefiable dense layer the settlements are within the limits of 10cm, which remain in the same limits with the liquefiable centre layer thickness of 2-4m. The settlements exceed the limit of 10cm when the thickness of the centre liquefiable layer is 8-12m.

The results suggest that due to the presence of the liquefiable layer in the centre of the soil profile, the settlement exceed the tolerable limits and as the thickness of that layer increases the settlement further increases. The settlements exceed the limits of 10cm when the thickness of the centre liquefiable layer is too large. In this situation, the tips of the columns (lying inside that liquefiable soft layer) are at a relatively larger distance from the top of the base non liquefiable dense layer; therefore do not get support from the base non liquefiable dense layer.

In fourth series, the beneficial effect of presence of non liquefiable dense layer in the centre of three layer soil profile between surface and base liquefiable layers on the treatment performance was studied by varying its thickness as shown in Fig. 15.

The results for A and B type geometries, as shown in Fig. 16, illustrate that for the full depth liquefiable layer the settlements exceed the limits of 10cm. For A type geometry, the presence of the 8 m non liquefiable layer in the centre of the soil profile reduce the settlements to the limits of 10cm, which remain in the same limits with a larger thickness. For B type geometry, although settlements decrease with increase in thickness of centre non liquefiable layer dense, they still exceed the limits.

The results suggest that the presence of a non liquefiable layer of any thickness in the centre of soil profile slightly decrease the settlements, but increasing thickness of that

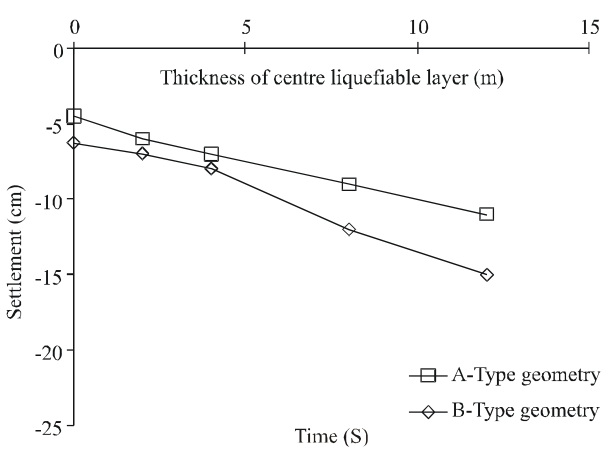

FIG. 14. SETTLEMENT OF FOOTING VERSUS THICKNESS OF CENTRE LIQUEFIABLE LAYER

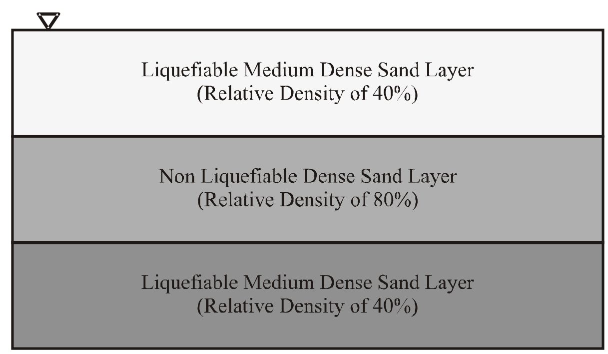

FIG. 15. THREE LAYER SOIL PROFILE WITH CENTRE NON LIQUEFIABLE LAYER

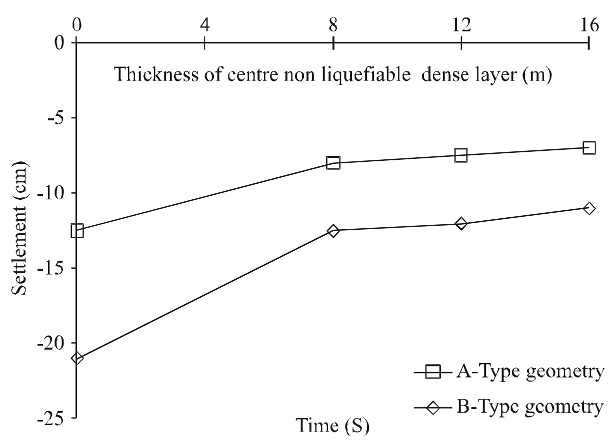

FIG. 16. SETTLEMENT OF FOOTING VERSUS THICKNESS OF CENTRE NON LIQUEFIABLE DENSE LAYER

layer has no significant effect on the further reduction of the settlements (settlements still exceed the tolerable limits) due to presence of liquefiable layers in the soil profile.

5. CONCLUSIONS

The following conclusions can be drawn from the present study:

(i) The settlements decrease with the increase in the length of columns. The settlements are relatively smaller when tips of columns are near or on the top of base non liquefiable layer. When it is imperative to limit the settlements to the tolerable limits, the lengths of columns should be extended by 2m in the base non liquefiable layer. Further increases in length give minor relative benefit.

(ii) For two layer soil profiles with surface liquefiable layer, the settlement remains in the tolerable limits as long as the thickness of that surface liquefiable layer is so thin that columns are extended by at least two metres in base dense layer. The settlements increase when the liquefiable layer is thick.

(iii) In a two-layer soil profile, presence of a surface non liquefiable dense layer as a foundation layer of relatively smaller thickness has no effect on the improvement of the settlements unless the thickness of that surface layer is large enough that tips of columns embed inside that dense layer to a certain the depth and even in that case settlements could not be reduced to tolerable limits (only to the limits of 10cm) due to presence of base liquefiable layer.

(iv) In the three layer soil profile, due to presence of the liquefiable layer in the centre of profile settlements exceed the tolerable limits and as the thickness of that layer increases the settlements further increase.

(v) In the three-layer soil profile, with the non liquefiable layer of any thickness in the centre of the soil profile, settlements slightly improve but there is no further significant effect on the reduction of settlements with an increase in the thickness of that layer and settlement still exceeds tolerable limits.

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