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3D Numerical Modelling of Liquefaction-induced Settlements and its Mitigation

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Abstract: In this paper, the behaviour of a 3D soil-structure system of square footing pad constructed on the ground reinforced with jet grouted columns was studied with 3D models using the commercially available FLAC 3D version 3.1 finite difference Code. The isolated shallow square footing pad supporting a typical building was constructed on the ground susceptible to liquefaction and on the reinforced ground with stiff jet grouted column rows. The results showed that ground reinforced with one row of continuously overlapping columns (forming a lattice) adjacent to the four sides of footing pad can be effectively applied to reduce the liquefaction-induced large settlements to the tolerable limits.

Keywords: Liquefaction; Jet grouted columns; FLAC 3D, Model, Numerical Modelling

1. INTRODUCTION

The buildings in earthquake-prone areas of Pakistan are susceptible to damage due to liquefaction-related large settlements. These large settlements are due to bearing capacity failure (shear failure) as the stiffness of soil is lost because of liquefaction. The ground reinforcement with stiff high modulus jet grouted/ deep mixing column rows which has performed well in earthquakes such as Kocaeli, Turkey, 1999 and Kobe, Japan, 1995 (Martin *et al.*,2004; Mitrani *et al*., 2008) is cost effective and easy to install in the ground.

In the previous studies (Almani *et al.*, 2012a) , the behaviour of shallow footing founded on the ground reinforced with jet grouted columns was studied in 2D plane strain, which represents the long strip footing pads in the direction perpendicular to the plane of analysis. With this assumption, deformations (strains) of the soil-structure system occur in the plane of analysis, while the deformations (strains) in the direction perpendicular to the plane of analysis were assumed to be negligible. Therefore, the horizontal boundaries are assumed to be fixed in the direction perpendicular to plane of analysis. Furthermore, with this assumption, the footing was considered to be a strip of infinite length and the rows of columns were supposed to be long and continuous (overlapping to form walls) in the direction perpendicular to the plane of analysis.

In this study, the behaviour of a 3D soil-structure system consisting of a shallow square footing pad supported on the ground reinforced with jet grouted columns was studied by constructing 3D models using the commercially available FLAC 3D version 3.1 finite difference code (Itasca, 2009). In this way, the effects due to the assumptions taken in the FLAC 2D plane strain model on the response could be eliminated which are more significant in the 3D parameters such as geometry of treatment, structural foundations and earthquakes.

2. MATERIALS AND METHODS

The materials and methods are presented in the following sections.

Model development

A three dimensional rectangular mesh was developed for this analysis as shown in **(Fig.1).** The mesh consisted of eight-nodded brick elements, with every node in the mesh connected to four other nodes, with the exception of nodes at the boundaries of the mesh.

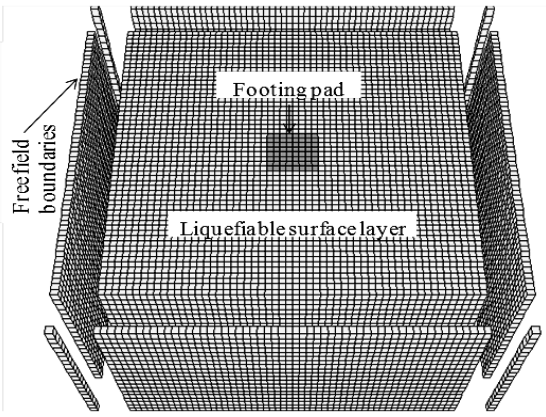


Fig. 1: Finite difference mesh for FLAC 3D analysis

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Mesh density

In order to get optimum mesh density (making the mesh as fine as possible keeping the aspect ratio near to 1 and simulation time), three meshes of zone size 0.25 m x 0.25 m x 0.25 m, 0.5 m x 0.5 m x 0.5 m and 0.5 m x 0.5 m x 1 m size were initially tested. The results showed that the response of the soil-structure system was identical, particularly the value of the settlements and the pore pressures which are the variables of primary interest for this study. Further, simulation times of the finer mesh of 0.25 m x 0.25 m x 0.25 m and 0.5 m x 0.5 m x 0.5 m were too long with this dynamic and coupled with a ground water flow problem (due to the very small combined dynamic and flow timestep). In view of the above, a mesh of 0.5 m x 0.5 m x 1 m zone size with an aspect ratio of 1 in the horizontal direction and 2 in the vertical direction, for which a simulation time was about 72 hours, was taken for this study.

Mechanical and hydraulic boundaries

The boundaries of the model were located in a way that the responses of the structure soil systems in the area of interest are not affected. For this purpose, initially, the boundaries of the grid were located at a distance of 2.5 times (50 m) of the layer thickness on each side of the centre of the structure. To further optimise the mesh size, boundaries were located at a distance of 20 and 15 m from each side of the centre of the structure. The results showed that the response was not affected with closer boundaries of 15 m each side of the centre of the structure. On the basis of this, a 30 m x 30 m x 20 m mesh was finally selected.

For static equilibrium of the model, fixed lateral and bottom boundaries were applied. For dynamic simulation, free field boundaries were applied along four lateral boundaries of the model. For hydraulic boundaries, the pore pressures along the top boundary of the model were set at zero for a free drainage surface.

Numerical modelling code selection and coupling of modules

The computer code FLAC 3D Version 6.0 (for Fast Lagrangian Analysis of Continua) was chosen for numerical analysis. Liquefaction problem could be modelled with this code by coupling the dynamic module with ground water flow module. For more details, the FLAC User's Manuals Itasca Consulting Group, 2009.

Basic soil properties, soil liquefaction model and damping

The Finn/Byrne liquefaction soil model in FLAC 3D for modelling the phenomenon of liquefaction is based upon Mohr-Coulomb failure criteria in conjunction with hysteric model as described in Almani *et al.,* 2012a. Soil properties are given in (**Table-1)**

Table 1: Properties of soil layers

Property	Soil layers	
	Medium	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)	2 x 10^{-7}	1 x 10^{-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 (Υ_{ref})	0.05	0.05
Water Bulk modulus (kPa)	5 x 10^5	5 x 10^5
Water tension (kN/m)	1 x 10^2	1 x 10^2
Water density (kg/m^3)	1000	1000

The Finn/Byrne soil model as described by relationship-1 was calibrated by simulating the constant volume cyclic simple shear tests single element test as described in detail in Almani *et al*., 2012a. The model parameters C_1 and C_2 and the Hardin-Drnevich hysteretic damping strain constant (Υ_{ref}) determined by calibration are shown in **(Table 2)**

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

In the above relationship, $\Delta\varepsilon_v$ 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 soil model constants.

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
Υ_{ref}	0.05	0.05

Ground reinforcement

The ground was reinforced with stiff jet grouted/deep mixing circular column rows of 0.6 m diameter (or 0.5 m x 0.5 m square columns with cross-sectional area equal to circular columns. The length of columns was set as 11 m. The column jet grouted material (cemented sand) was represented with the Mohr-Coulomb soil model combined with Hardin–Drnevich hysteretic dynamic damping model as described in Almani *et al.,* 2012b.

2. RESULTS AND DISCUSSIONS

The square footing pad of 4 m x 4 m x 1 m size founded on liquefiable ground at the depth of 1 m from the ground surface as shown in **(Fig. 2),** was modelled as 3D dynamic coupled with ground water flow. This is a base (benchmark) case for comparison.

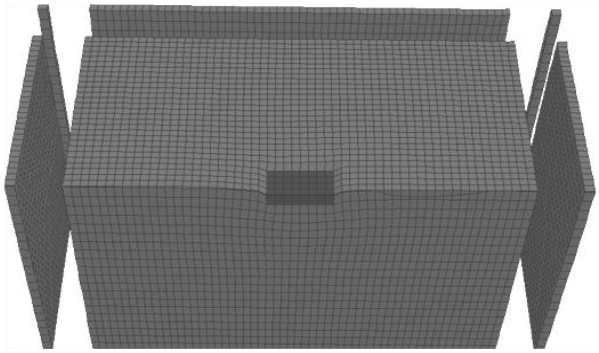


Fig. 2 : The settlement of the footing-Benchmark case

The history and contours of the settlement at the centre of the footing pad, as presented in **Fig. 3** and **Fig.** , shows that the settlement of the footing pad is as large as 0.7 m (vertical displacement of 70 cm) at the end of 10 seconds of cyclic loading. This behaviour of the soil-structure system was identical to the predicted with the FLAC 2D plane-strain model (Almani *et al.* 2012a), though the settlements observed in the 3D case are smaller in magnitude than its counterpart in 2D due to the assumption of strip long footing pad in the case of the 2D plane-strain model (Almani *et al.*, 2012a).

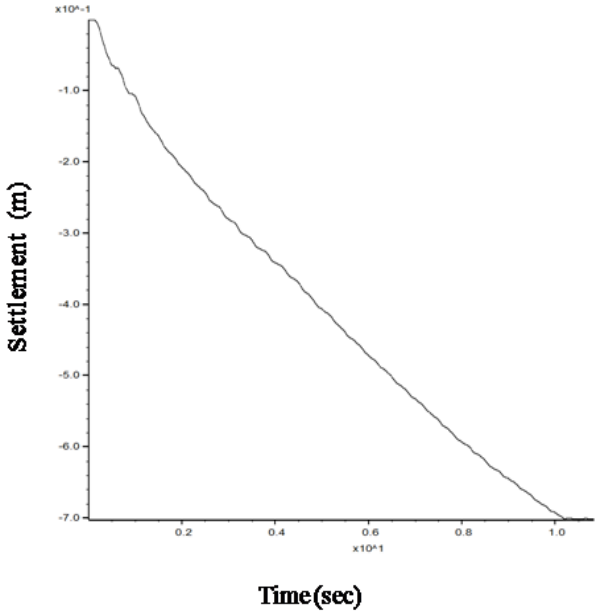


Fig. 3: The settlement of footing vs. time-Benchmark case

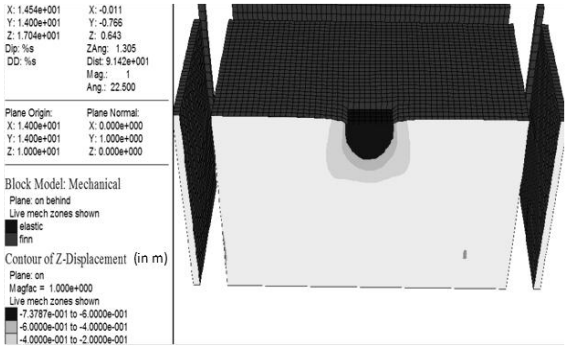


Fig. 4: Contours of vertical displacement-Benchmark case

The history of pore pressure at the 4 m depth under the centre of the footing pad, as presented in Fig., shows that the pore pressure increases from the initial hydrostatic value of $43x10^3$ Pa to the peak value of $138x10^3$ Pa in the initial cycles of cyclic loading, which soon decreases to $60x10^3$ Pa due to the effect of dilation (expansion) caused by monotonic shearing under the influence of the structure. This pattern of pore pressure variation is the same as predicted with the 2D plane-strain model (Almani *et al*, 2012a).

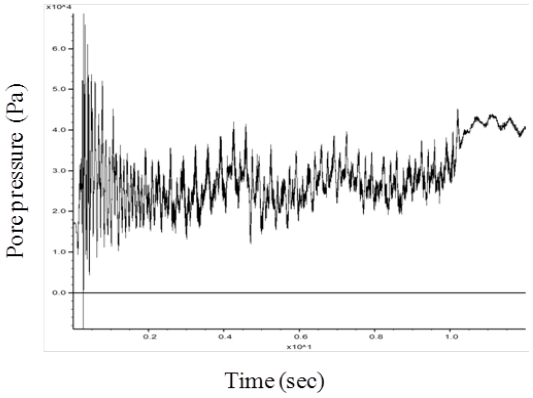


Fig. 5. Pore pressure vs. time under the structure-Benchmark

In the other study, isolated shallow square spread footing 4 m x 4 m x 1m size was founded on the ground reinforced with continuously overlapping column rows forming one lattice beneath and two lattices at 0.5 m spacing around the footing pad in both directions, as shown in **(Fig. 6).**

The histories of the settlement at the centre of the footing pad, as presented in **(Fig. 7)** shows that the settlement of the footing is within the tolerable limits of 0.04 m (4 cm) when the dynamic excitation is applied for 10 second duration as against the 0.7 m (70 cm) in the benchmark case. However, there is slight heave or rebound of footing due to transmission of motion through the stiff the columns to the base of footing from the base of the model in this reinforced case.

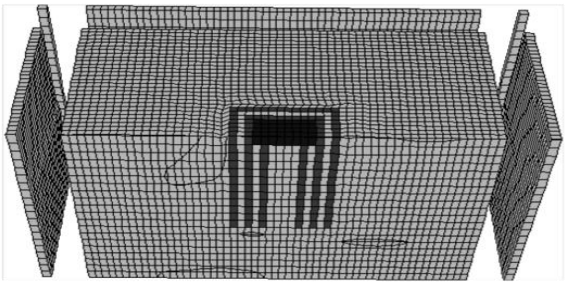


Fig.2: Ground reinforced with rows of jet grouted columns

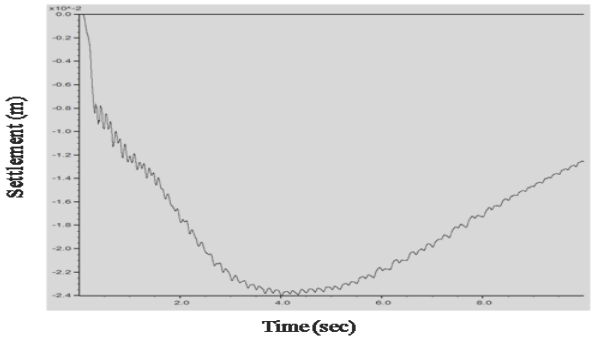


Fig. 3: Settlement of footing vs. time-Reinforced case

The history of pore pressure at 4 m depth under the centre of the footing pad, as shown in **(Fig. 8),** indicates that at the centre of the pad in the untreated soil zone, the pore pressure increases from the hydrostatic value of $45x10^3$ Pa to the peak value of $100x10^3$ Pa during initial the cycles of the dynamic event (as against its peak value of $138x10^3$ Pa in the unreinforced benchmark case). This pattern of pore pressure variation under the structure is the same as observed in 2D in the initial cycles of loading, but in later cycles pore pressure in the 3D model is relatively higher in magnitude than the counterpart 2D reinforced model case (Almani *et al.*, 2012b).

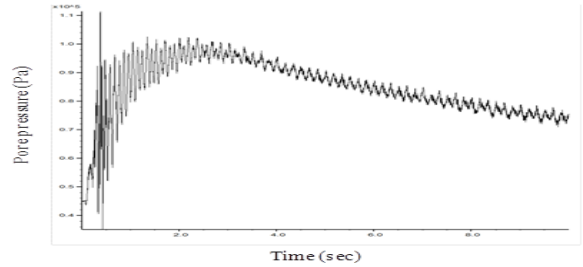


Fig. 4: Pore pressure vs. time under footing-Reinforced case

These results confirm the 2D results that when ground is reinforced at certain spacing with rows of jet grouted columns, the peak pore pressure in the initial cycles decreases by 30% as compared to base benchmark. This decrease of pore pressure is not significant that liquefaction does not occur and settlements could be prevented.

5. CONCLUSION

The following conclusion can be drawn from the above studies:

- 3D shallow square footing settles by 70 cm in the punching type bearing capacity (shear) failures when the foundation soil liquefies when pore pressures under the footing quickly rise in the few cycles.
- By reinforcing the ground with jet grouted columns under the structure, pore pressures somewhat decrease, but are still high enough to cause significant liquefaction under the footing.
- Ground reinforced with one row of continuously overlapping columns (forming a lattice) adjacent to the four sides of footing pad and two rows of continuously overlapping columns in both direction (one lattice) beneath the footing pad, or rows of continuously overlapping columns forming one lattice beneath and one lattice adjacent to the footing pad, can be effectively applied to meet the tolerable settlement limits.

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