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Effect of Earthquake Characteristics on Liquefaction Resistance of Reinforced Ground

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Abstract : In this paper, the effect of characteristics of earthquake on the behaviour of shallow foundations on the ground reinforced with jet grouted columns was analysed using numerical modelling (FLAC 2D). The ground reinforcement by installing stiff jet grouted columns under the shallow foundations of building was applied as liquefaction remediation measure when the foundation soil is expected to liquefies during an earthquake. The isolated shallow square footing pad supporting a typical simple frame structure was constructed on the reinforced ground with stiff jet grouted column rows at the shallow depth from the ground surface. This soil-structure system was modelled and analyzed as plane-strain using the FLAC 2D dynamic modelling and analysis Software. The results showed that settlement increases nearly linearly with the peak amplitude of the earthquake. The settlements are generally larger (within the 10 cm limit) at a lower range of frequencies (0.25 to 2Hz) and smaller (within tolerable limits of 5 cm) at a higher range of frequencies (3 to 12Hz). The settlements increase slightly with an increase in duration of shaking event.

Keywords: Liquefaction; Jet grouted columns; Amplitude; Frequency, Duration, Numerical modelling.

1. INTRODUCTION

Karachi, the biggest commercial hub of Pakistan is lying in the region most vulnerable to earthquakes which usually originate from epicentre at Gujrat fault. In coastal areas of Karachi, Defence Housing Authority (DHA) and Clifton area soil deposits are susceptible to liquefaction in the event of earthquake. The buildings in these areas may be damaged due to liquefaction-related large settlements. These large settlements are initiated due to bearing capacity or shear failure when the soil loses stiffness as a result of liquefaction and also due to reconsolidation (densification) volume change of the soil occurs when pore water is dissipated. 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, 1999 and Kobe, Japan, 1995 (Martin *et al.*,2004; Mitrani *et al.*, 2008) may be more cost effective and easier to install. Further, with relatively less replacement area, motions transmission towards structures may be relatively less.

The settlement and its mechanisms strongly depend on key design parameters, such as the initial relative density (D_r) of liquefiable soil layers; type and load of structure and geometry and the characteristics of input ground motion such as peak ground acceleration PGA (Dashti *et.al*, 2010). In this regard, the effect of these varying design parameters such as type of structure, soil profile and earthquake characteristics on treatment performance is particularly important. Therefore, in this study, the effect of varying earthquake characteristics such as amplitude, frequency and duration on settlement of shallow foundation constructed on the ground reinforced with jet grouted columns was analyzed.

2. MATERIALS AND METHODS

For this study, the isolated footing of 4 m x 4 m pad size and 1 m thickness was constructed at 1 m depth from the ground surface. This footing supports a central column of 1 m x 0.5 m cross-section and 5 m in length, which supports a part of superstructure, as shown in **(Fig.1).**

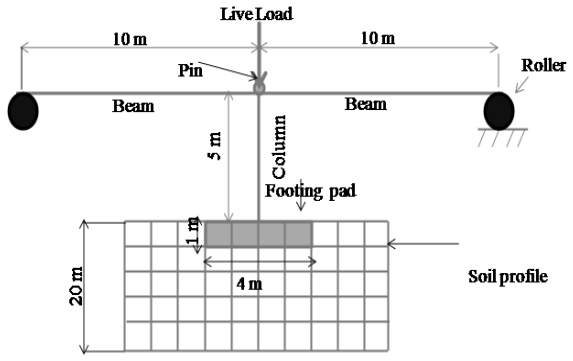


Fig.1: Superstructure and footing of 4 m wide x 1 m deep (Mesh and structure not drawn as per scale but shows model qualitatively)

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The soil profile with two layers shown in **(Ошибка! Источник ссылки не найден.),** was taken for this study. This soil deposit consists of liquefiable medium dense (LB) E-Fraction Leighton Buzzard silty sand layer with thickness of 10 m as the surface layer (at 40 % relative density) underlying which non liquefiable dense layer (at relative density of 80%) with thickness of 10 m. Further details for structure, its properties and soil profile have been described in detail in (Almani *et al., 2012a).*

2.1 Numerical modelling code selection and coupling of modules

The computer code FLAC 2D Version 6.0 (for Fast Lagrangian Analysis of Continua) was chosen for numerical analysis. Liquefaction problem can be 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 Itasca Consulting Group, 2009).

2.2 Basic soil properties, soil liquefaction model and damping

The Finn/Byrne liquefaction soil model in FLAC 2D for modelling the phenomenon of liquefaction is based upon Mohr-Coulomb failure criteria in conjunction with hysteric model (Manuals Itasca Consulting Group, 2009). This combined constitutive model presents the liquefaction behaviour of soil 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 and Idriss, (1970) as described in (Almani *et al.,* 2012a). The soil properties are given in **(Table 2).**

Table 1: Properties of soil layers

Property	Soil layers	
	Medium	Dense
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

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 more detail in (Almani *et al*.,2012a). The model parameters C_1 and C_2 and the Hardin-Drnevich hysteretic damping strain constant (Υ_{ref}) as shown in **(Table 2).**

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

In this equation, $\Delta\varepsilon_v$ 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. Υ_{ref} for the Hardin-Drnevich hysteretic dynamic damping model was taken as 0.05 for sand, as recommended in the FLAC 2D manual (Manuals Itasca Consulting Group, 2009).

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

2.3 Ground reinforcement

The ground was reinforced with stiff jet grouted/deep mixing circular column rows of a relatively same small diameter of 0.6 m (or 0.5 m x 0.5 m 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) as described in (Almani *et al.,* 2012b).

2.4 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. . 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 4 cm recommended by Skempton and MacDonald, (1956). These are the design limits for maximum settlements up to which building is in serviceable condition.

3. Limits of 10 cm at which the structure loss serviceability but may not collapse (Duncan and Tan, 1991).

3.1 Model development

For this study, the rectangular mesh with uniform zone size of 0.5 m x 0.5 m and the aspect ratio of one (1) was made as shown in **(Ошибка! Источник ссылки не найден.).** The fixed lateral and bottom boundaries were applied during the static analysis whereas during dynamic analysis (earthquake loading), free field boundaries were applied at the two lateral boundaries of the model. The lateral boundaries were of the model were taken at far distance so that the behaviour of the soil-structure system (stresses, strains and pore pressure) in the area of structure and surrounding soil is not influenced due to the effects of boundaries. In this case the model was made wide so that lateral boundaries are three times of the soil profile depth.

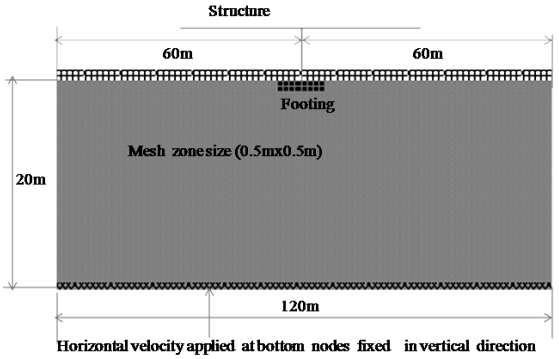


Fig. 2: Finite difference mesh used for FLAC 2D numerical analysis

In order to mimic hydraulic boundaries, the pore pressures for the top drainage boundary were set as zero (0) to simulate that drainage surface. The lateral and bottom boundaries of the model were taken as impervious.

4. RESULTS AND DISCUSSION

The characteristics of an earthquake can be quantified with peak amplitude, dominant frequency and duration. In order to predict the response of the soil-structure system in more severe and rare events of earthquakes than design earthquake is important for effective design. In this study, the effect of varying earthquake characteristics (amplitude, dominant frequency and duration) on treatment performance was studied. For this purpose, 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 The effect of amplitude of earthquake

In first study, the effect of the large amplitude of the earthquake (0.7g or larger) on treatment performance was studied by increasing the amplitudes up to 0.6 m/sec for the standard sinusoidal velocity wave with a typical earthquake frequency of 3.0 Hz. for 10 sec duration.

The results for A type geometry, as shown in **(Fig. 3),** reveal that the settlement at the centre of the footing pad is within the tolerable limits of 5 cm with the velocity amplitude of 0.2 m/sec, which increases to limits of 10 cm with higher amplitudes of 0.4 and 0.6 m/sec. The settlement exceeds the limits of 10 cm when the amplitude of the earthquake is as large as 0.8 m/sec.

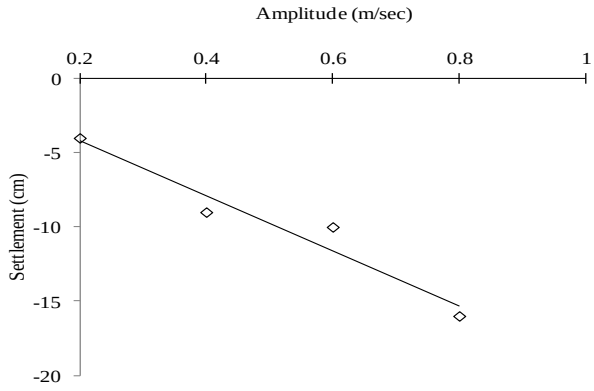


Fig. 3 : Settlement of footing vs. amplitude of velocity wave-A-type geometry

The results for B type geometry, as shown in **(Fig. 4),** reveal that the settlement at the centre of the pad is within the limits of 10 cm at the amplitude of 0.4 m/sec. The settlement exceeds the limits of 10 cm when the amplitude is 0.6 m/sec and larger. The results for C type geometry, as shown **(Fig. 5),** reveal that the settlement at the centre of pad is within the tolerable limits of 4 cm when the amplitude is 0.2 m/sec, which increases to the limits of 10 cm when the amplitude is 0.4 and 0.6 m/sec. The settlement exceeds the limits of 10 cm when the amplitude is as large as 0.8 m/sec.

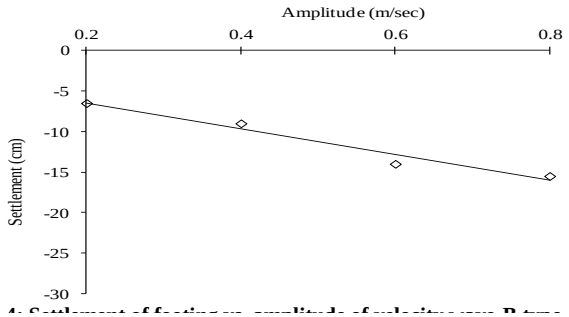


Fig. 4: Settlement of footing vs. amplitude of velocity wave-B type geometry

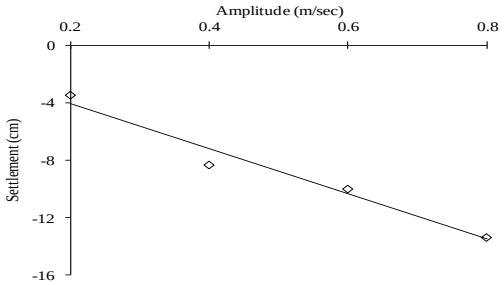


Fig. 5: Settlement of footing vs. amplitude of velocity wave-C type geometry

The above results suggest that the settlement increases with the amplitude of the earthquake. This increase in settlements exceed the tolerable limits for very severe earthquakes with a peak velocity amplitude of 0.4 m/sec or a ground acceleration of 0.7g (peak acceleration amplitude of 7 m/sec) and larger.

This increase in the settlements with amplitude can be explained by the comparison of shear and volumetric strains at the larger and smaller velocity amplitude of 0.8 and 0.2 m/sec, as presented in **(Fig.6 to Fig. 9).** As shown, shear and volumetric strains are 5% and 0.3% at 0.8 m/sec as against the 1% and 0.1% at 0.2 m/sec respectively. The more shearing of the soil under the structure at higher amplitude of earthquake develops more pore pressure which result more softening in soil under the structure. In addition, higher development of pore pressure causes higher pore pressure gradients which causes larger volume change in the all regions of soil profile (globally) in the later cycles of dynamic excitation due to faster drainage of water towards the free surface (as evident from larger volumetric strains at larger amplitude). Both these phenomena cause larger settlements as the amplitude of earthquake increases.

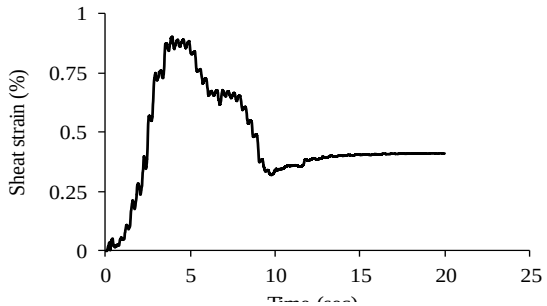


Fig.6: Shear strain vs. time at 0.2 m/sec velocity amplitude

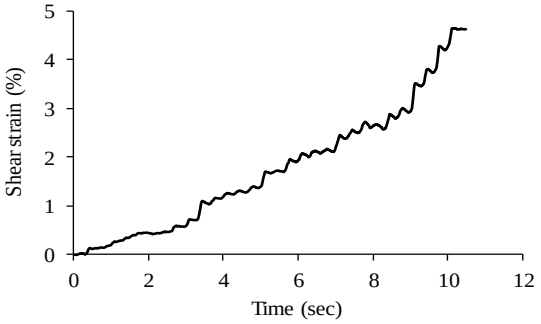


Fig.7: Shear strain vs. time at 0.8 m/sec velocity amplitude

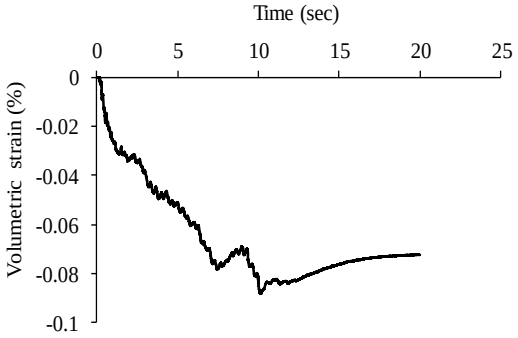


Fig.8: Volumetric strain vs. time at 0.2 m/sec velocity amplitude

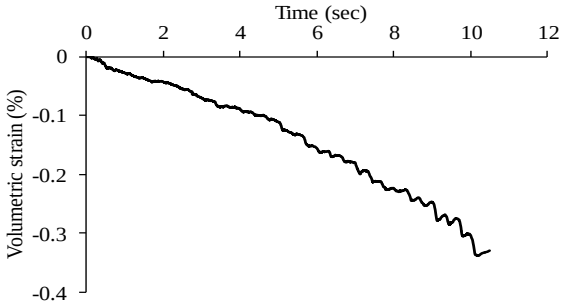


Fig. 9: Volumetric strain vs. time at 0.8 m/sec velocity amplitude

4.2 The effect of frequency of earthquake

In other series of models, the effect of the dominant frequency of earthquake on the treatment performance of adjacent type (A type) geometry was studied. Frequencies ranging from as low as 0.25 Hz. to as high as 12 Hz. were applied in the standard sinusoidal velocity wave with an amplitude of 0.2 m/sec for a duration of 10 seconds at the base of model, as shown in **(Fig.10).** In order to separate the effects of frequency from the effects of the number of cycles on the treatment performance, the cyclic loading was applied for

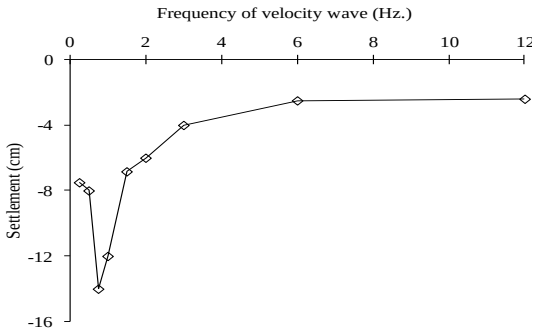


Fig.10: Settlement of footing vs. frequency of velocity wave

The results show that at low range of frequencies between 0.25 to 2.0 Hz. the settlements are larger (with limit of 10 cm) but at higher range between 3.0 to 12 Hz, the settlements are smaller (within tolerable limits of 4 cm). The settlements exceed the limits of 10 cm at the frequencies of 0.75 and 1 Hz.

The history of the velocity wave at the base of footing as presented in **(Fig.11)** shows that the velocities are amplified relative to the velocity amplitude at the base of model as shown in **(Fig.12)** which indicates the effects of resonance of system. The large settlements at these frequencies are due to this effect of this resonance when the natural frequency of soil-structure system matches or is near to the frequencies of the earthquake which causes amplification of motions. Due to these resonance effects, settlement increases drastically.

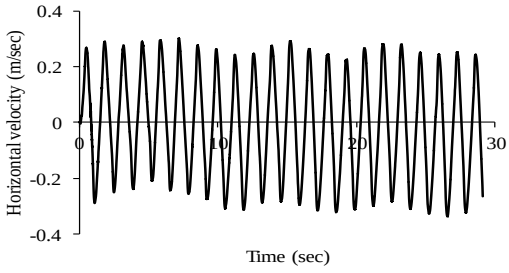


Fig.11: Amplification of velocity amplitudes at base of footing at resonance

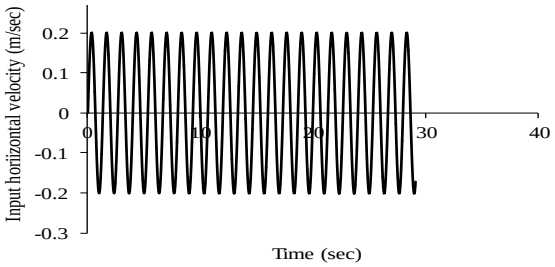


Fig.12: Input horizontal velocity at the base of model

The above results suggest that for the frequencies of the earthquake which match with the natural frequencies of the system, the settlements may be large which may be critical for the structure system. At other frequencies of earthquake at which the effects of resonance are not significant, the settlements are smaller.

4.3 Effect of duration of earthquake

In other study, the effect of the duration of the earthquake on treatment performance of adjacent type (A type) geometry was studied. The cyclic loadings of varying durations, ranging from a small duration of 2.5 seconds to a large duration of 20 seconds (equal to 7.5 to 60 cycles of loading), were applied in the velocity sinusoidal standard wave at the amplitude of 0.2 m/sec and with the typical frequency of 3.0 Hz., as shown in **(Fig.13).**

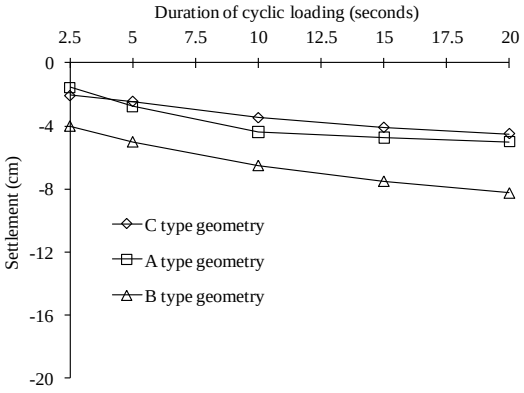


Fig.13: Settlement of footing vs. duration of cyclic loading

For A type geometry, results show that the settlement is within the tolerable limits of 4 cm when the dynamic excitation is applied for the small durations of 2.5 and 5 seconds (7.5 and 15 cycles), which slightly increases to the tolerable limits of 5 cm when the cyclic excitation is applied for longer durations of 10 to 20 seconds.

For B type geometry, results show that the settlement is within the tolerable limits of 5 cm when the dynamic excitation is applied for the small durations of 2.5 and 5 seconds (7.5 and 15 cycles), which increases to the limits of 10 cm when the cyclic excitation is applied for longer durations from 10 to 20 seconds.

For C type geometry, results show that the settlement is within the tolerable limits of 4 cm when the dynamic excitation is applied to the durations of 10 seconds (30 cycles), which increases slightly to the tolerable limits of 5 cm when cyclic excitation is applied for

The results show that the settlements slightly increase but meet the tolerable limits (expect for B type geometry). This slight increase of the settlements is due to additional reconsolidation (volume change) of soil in the later cycles of cycles loading when more (additional) pore pressure is developed and dissipated with the longer cyclic shearing with longer duration shaking. This dissipation of pore water for longer time (during cyclic loading) to the top drainage surface causes additional volume changes or reconsolidation of soil due to which the settlements increase.

5. CONCLUSIONS

The following conclusion can be drawn from the above studies:

1. Settlement increases linearly with the peak amplitude of the earthquake. In the event of design earthquakes with peak ground acceleration of 0.35 g, the settlements meet the tolerable limits. However, settlements exceed the tolerable limits for extremely severe earthquakes.

2. The settlements are generally larger (within the 10 cm limit) at a lower range of frequencies (0.25 to 2Hz) and smaller (within tolerable limits of 5 cm) at a higher range of frequencies (3 to 12Hz). However, for frequencies of the earthquake which match with the natural frequencies of the system, the settlements may be large due to the effects of resonance which may be critical for the soil-structure system.

3. The settlements increase slightly with an increase in duration of shaking but remain in the tolerable limits.

6. ACKNOWLEDGEMENTS

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