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Numerical simulation of pile group behaviour in liquefying sloping ground

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NUMERICAL SIMULATION OF PILE GROUP BEHAVIOUR IN LIQUEFYING SLOPING GROUND

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ABSTRACT: This paper investigates pile group behaviour in liquefying sloping ground. The numerical procedure utilised involves two main steps. First a ground response analysis is carried out using a stress path model to obtain the maximum ground displacements along the pile and the degraded soil modulus over the depth of the soil deposit. Next a dynamic analysis is carried out for a single pile in the group assuming that each pile in the group behaves in the same way during the earthquake loading. The method has been verified using centrifuge data, where soil liquefaction has been observed in laterally spreading sloping ground. It is demonstrated that the new method gives good estimate of pile behaviour, despite its relative simplicity.

1. INTRODUCTION

Numerical simulation of pile group behaviour in liquefying sloping ground under earthquake loading is a complex problem. The loss of soil stiffness and strength due to excess pore pressure generation during earthquake loading may develop large bending moments and shear forces in piles, leading to pile damage. The major earthquakes that have occurred during past years such as the 1964 Niigata, 1989 Loma-Prieta and 1995 Hyogoken-Nambu have clearly demonstrated the significance of soil liquefaction-related damage to pile foundations. Therefore, currently there is a great demand for validated numerical procedures to predict pile behaviour in liquefying soil.

In this paper a Winkler model developed by Liyanapathirana and Poulos (2005) has been used to predict pile group behaviour in liquefying soil subjected to lateral spreading, where the Mindlin's equation has been utilised to determine the non-linear spring constants of the Winkler model. Depending on the amount of pore pressure development, the spring coefficients in the spring-dashpot model are degraded while the effect of radiation damping is taken into account separately.

The validity of the model has been verified by using the pile group behaviour observed during centrifuge tests carried out by Brandenberg et al. (2004) in liquefying sloping ground subjected to lateral spreading. It is demonstrated that the proposed method is accurate enough to predict the pile behaviour in liquefying soil for design purposes, despite its relative simplicity.

2. NUMERICAL MODEL

The one-dimensional numerical model developed for the analysis of pile groups in liquefying ground subjected to lateral spreading is based on the finite element method and involves two stages.

2.1 Free-field ground response analysis

The ground response analysis is carried out by dividing the soil deposit into a number of layers. The soil is modelled using a hyperbolic stress-strain relationship, which reflects non-linear, strain dependent and hysteretic behaviour of the soil. Pore pressure generation is calculated based on the stress path model described in the companion paper Liyanapathirana (2007).

2.2 Pile analysis

Pile analysis is carried out based on the method proposed by Liyanapathirana and Poulos (2005), where the interaction between the soil and the pile is modelled using the analysis method for a dynamically loaded beam on a non-linear Winkler foundation. The pile is modelled as a beam, and the lateral pressure acting on the pile is modelled using a spring-dashpot model. The partial differential equation for a beam on Winkler foundation is given by,

$$E_P I_p \left(\frac{\partial^4 U_p}{\partial z^4} \right) + M_p \left(\frac{\partial^2 U_p}{\partial t^2} \right) = Kx\left(U_{ff} - U_p\right) + C_x \left(\frac{\partial U_{ff}}{\partial t} - \frac{\partial U_p}{\partial t} \right) \quad (1)$$

where E_p is the Young's modulus of the pile material, I_p is the inertia of the pile, U_p is the pile displacement, U_{ff} is the free-field lateral soil displacement, M_p is the mass of the pile, and K_x and C_x are the spring and dashpot coefficients of the Winkler model. A solution to the problem can be obtained by solving Equation (1) using the finite element method.

The Mindlin hypothesis does not include the soil radiation damping and this should be incorporated into the analysis separately. In previous studies, a value of $5\rho_s V_s$ has been used for the dashpot coefficient (Liyanapathirana and Poulos, 2005). However, during the comparison of numerical results with centrifuge data it has been found that $5\rho_s V_s$ is extremely high for laterally spreading sloping ground considered in this study. Therefore, a reduced value of $\rho_s V_s$ is used here for the dashpot coefficient. This dashpot takes into account the radiation damping of the shear waves travelling away from the pile.

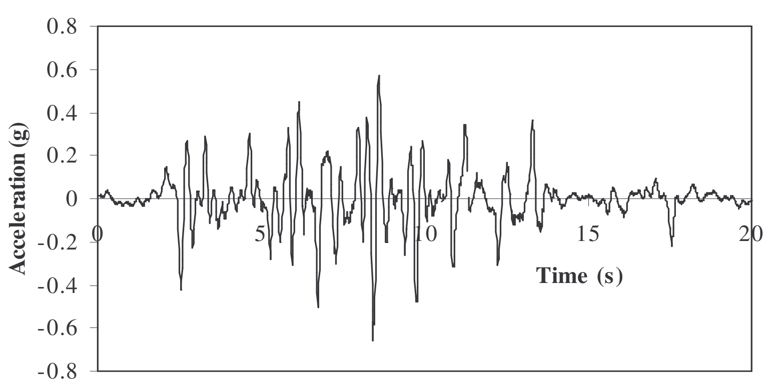

Figure 1. Santa Cruz motion scaled to 0.67g.

3. COMPARISON WITH CENTRIFUGE TEST RESULTS

Here the validation of the numerical model is carried out using the centrifuge tests carried out by Brandenberg et al. (2004) for pile groups founded on laterally spreading sloping ground. The soil profile in the centrifuge model consisted of a 1.4 m thick layer of Monterey sand, overlying a 2.8 m thick deposit of heavily overconsolidated Bay Mud, overlying a 5.6 m thick layer of loose Nevada sand (Dr = 35%), overlying 18.2 m thick layer of dense Nevada sand (Dr = 75%) in prototype scale. All soil layers were built to a slope of 3^0.

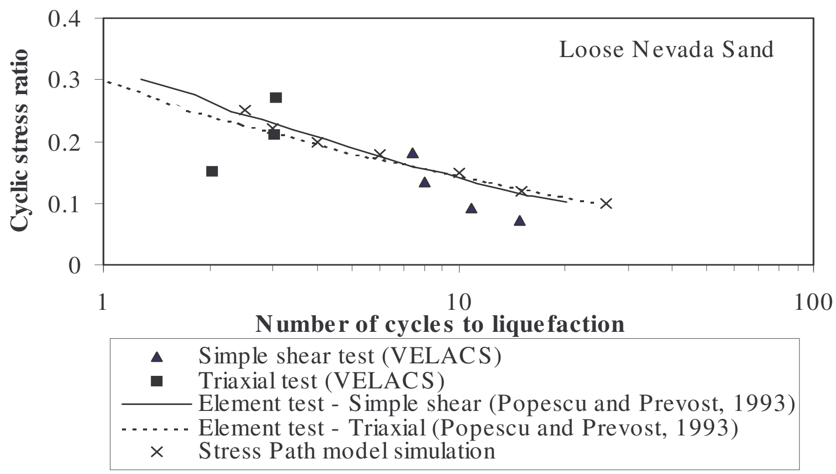

Figure 2. Cyclic shear strength simulated from the stress path method.

3.1 Calibration of the stress path model

The pile group consisted of six pipe piles. In the prototype scale, the piles had an outer diameter of 1.0 m, a wall thickness of 0.05 m, Young's modulus of 200 GN/m^2 and a density of 7500 kg/m^3. The cap mass was shared between the six piles. Hence each pile in the group carried a capmass of 100 t. The embedded length of the pile was 25.2 m. During the centrifuge test, the model was subjected to the Santa Cruz motion scaled to a maximum acceleration level of 0.67g. Figure 1 shows the input acceleration record.

The stress Path model has been calibrated using the cyclic shear strength data given by Popescu and Prevost (1993) for loose and dense Nevada sand. Figure 2 shows the cyclic shear strength curve for the the loose Nevada sand obtained from the numerical model, one element simulations by Popescu and Prevost (1993) and experimental data (VELACS). The stress Path model agrees well with the other data.

3.2 Ground response analysis for the sloping ground

Before the application of earthquake load, sloping ground is subjected to a static shear stress equivalent to the component of effective overburden stress parallel to the slope of the ground. In this case, the ground slope is 3^o. Therefore an initial static shear stress of $\sigma'_{vo}\, sin(3)$ has been applied to the ground. In sloping ground, displacements can become very large before the onset of soil liquefaction due to this static shear stress acting on the ground.

Shear modulus of the soil is represented by (Ishihara and Towhata, 1982):

$$G = G_o \frac{(2.17 - e)^2}{1 + e} \left(\frac{1 + 2K_o}{3} \sigma'_v \right)^{0.5} \text{ N/m}^2 \tag{2}$$

where e is the void ratio of soil and K_0 is the earth pressure coefficient at rest. The constant G_o is determined by matching the shear wave velocities measured during the centrifuge test for dense and loose Nevada sand layers. Pore pressure generation has been considered only for the loose and dense Nevada sand layers. In addition to the hysteretic damping, 6% of critical damping has been applied independently to take into account the viscous damping of the soil.

Figures 3 and 4 show the pore pressure generation at the centre of loose and dense Nevada sand layers. As observed during the centrifuge test, the loose sand layer has liquefied about 10 sec after application of the earthquake load. Numerical model predicts higher pore pressure development during the early stages of loading than that observed during the centrifuge test. In the dense Nevada sand layer, large pore pressure reductions have been observed during the earthquake loading due to dilation of the soil. Although the numerical model does not have the ability to model dilation of the soil, the overall agreement between the measured and observed pore pressure distributions in both loose and dense Nevada sand layers is reasonable.

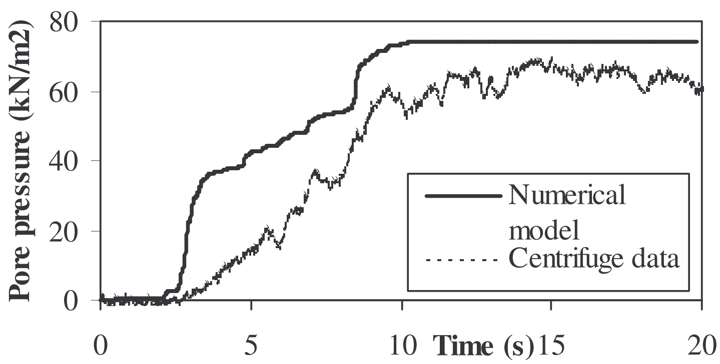

Figure 3. Pore pressure generation at the middle of the loose Nevada sand layer.

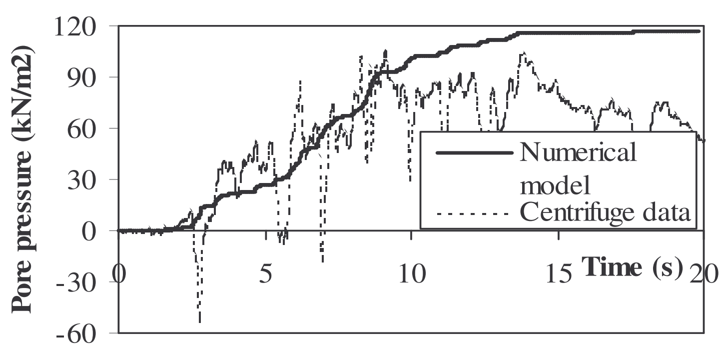

Figure 4. Pore pressure generation at the middle of the dense Nevada sand layer.

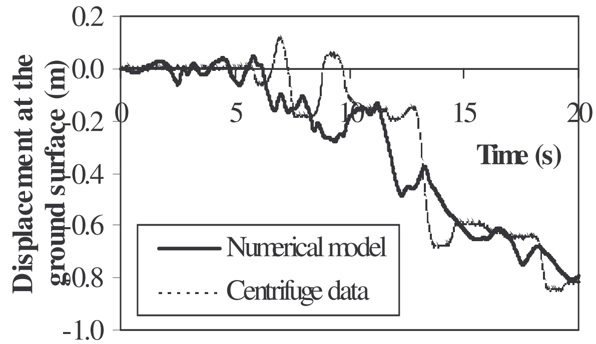

Figure 5. Displacement at the ground surface.

Figure 5 shows the ground displacement at the surface obtained from the numerical model and that measured closer to the pile group. The close agreement between those indicates that the influence of soil dilation does not have a significant contribution to the ground displacement.

3.3 Pile analysis

Since the pile cap height was nearly equal to the thickness of the layer of Monterey sand at the top of the model container, pile analysis was carried out only for the pile section below the Monterey sand layer. Although the pile group had six piles, in the one-dimensional pile analysis only one of them was considered, and it was assumed that all six piles behaved in the same way. According to the observed bending moment distributions shown in Figure 6 at z = -9.1 m for three different piles in the group, it can be seen that this assumption is reasonable to obtain pile behaviour for design purposes.

Table 1. Soil parameter values used for the analysis.

	Monterey sand	Bay Mud	Loose Nevada sand	Dense Nevada sand
Soil constant B'_u			2.0	1.0
Soil constant B'_p			1.0	0.4
Parameter k in pore pressure model			0.06	0.06
Earth pressure coefficient at rest, K_o	0.53	0.53	0.5	0.4
Void ratio, e	0.8	1.53	0.755	0.614
Density, ρ_s (kg/m^3)	1950	1580	1936.8	2018.35
G_o (N/m^2)	3.2x10^5	2.2x10^5	2.2x10^5	2.2x10^5
Friction angle, ϕ'	37	28	35	37
Undrained shear strength, c_u (N/m^2)	0.0	1.7x10^4	0.0	0.0

In the analysis, the lateral pressure at the soil-pile interface was monitored and an iterative procedure was used to keep it at or below the ultimate lateral presssure, P_y. When the lateral pressure at the pile-soil interface reaches the ultimate value, soil yielding occurs. For piles in clay, Broms (1964) has suggested that the ultimate lateral pressure varies from $2c_u$ to 9 c_u within top four diameters of the pile length and stays at 9 c_u below that level. A similar distribution was assumed here for the Bay Mud layer. For the two sand layers, $P_y = 0.3\sigma'_v$ was used (Japan Road Association, 2002). Figure 7 shows the bending moment distribution at z = -9.1 m obtained from the numerical model, and Figure 8 shows measured and computed maximum positive and negative bending moment distributions along the pile. It can be seen that the simplified analysis presented in this paper has the ability to predict pile behaviour with sufficient accuracy for design purposes.

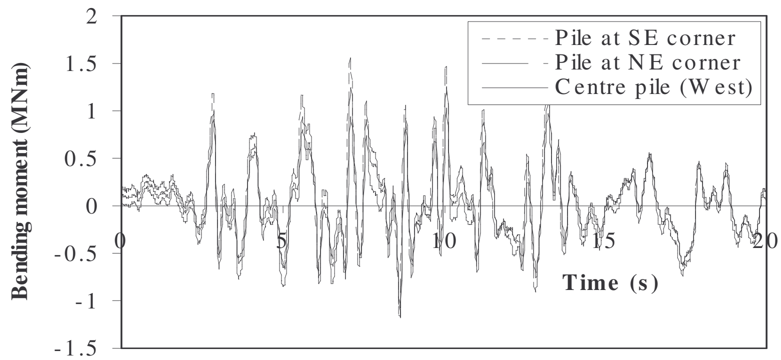

Figure 6. Pile bending moment close to the bottom of the loose sand layer (z = -9.1 m) observed for 3 piles in the group.

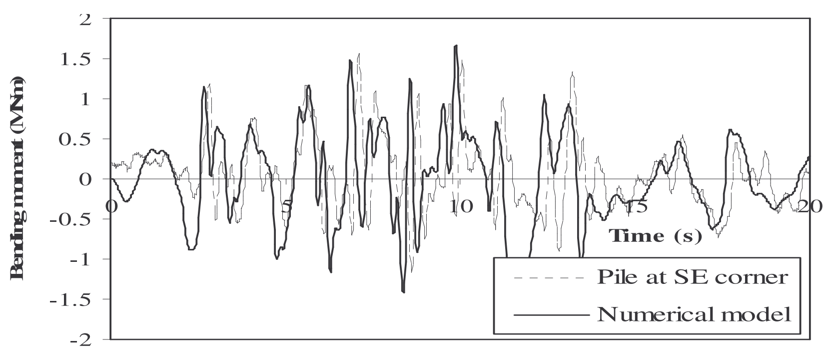

Figure 7. Pile bending moment close to the bottom of the loose sand layer (z = -9.1 m).

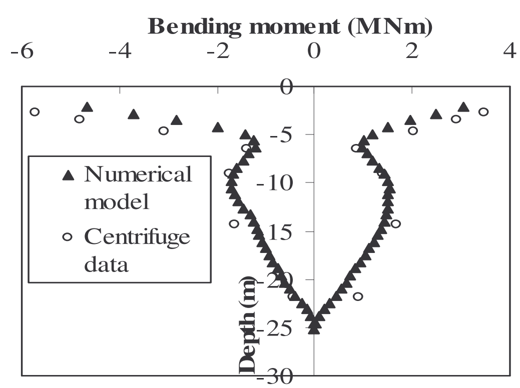

Figure 8. Maximum positive and negative bending moment distributions along the pile.

CONCLUSIONS

This paper has described a numerical procedure, which may be used to compute pile behaviour in liquefying soil subjected to lateral spreading. An effective stress based ground response analysis is first carried out and the resulting ground displacements and degraded soil stiffness are used to obtain the pile performance. The spring coefficients of the Winkler model are derived from Mindlin's equations. Pile performance observed during centrifuge tests has been simulated using the new method, and it is found that the pile response calculated from the new method is close to the observed behaviour. Thus, a reasonable estimate for piles in a group can be obtained by assuming that they all behave in the same way.

ACKNOWLEDGEMENTS

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