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Seismic Lateral Response of Piles in Liquefying Soil

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ABSTRACT: Soil liquefaction is one of the major factors affecting the behaviour of piles founded in seismically active areas. Although methods are available for seismic analysis of pile foundations, in many of them, the soil is assumed to be an elastic material. Here a numerical model is presented which takes into account the reduction of soil stiffness and strength due to pore pressure generation and subsequent soil liquefaction, in addition to the material non-linearity. Results obtained from the new method are compared with centrifuge test data and they show excellent agreement with the observed pile behaviour. To investigate effects of soil liquefaction on the internal pile response, a parametric study is carried out for a range of material and geometric properties of the pile and soil. The effect of the nature of earthquakes on pile performance has been studied using 25 earthquake records scaled to different acceleration levels. It is found that the 'pseudo velocity' of the earthquake, which is the gross area under the input acceleration record, has a significant influence on the pile performance in liquefying soil.

1 INTRODUCTION

The performance of piles in liquefying ground under earthquake loading is a complex problem due to the effects of a progressive build-up of pore water pressures in the saturated soils. The loss of soil strength and stiffness due to liquefaction may develop large bending moments and shear forces in piles, possibly leading to pile damage. The significance of liquefaction-related damage to pile foundations has been clearly demonstrated by the major earthquakes that have occurred during past years such as the 1964 Niigata, 1964 Alaska, 1989 Loma-Prieta and 1995 Hyogoken-Nambu events.

Although there remain many uncertainties in the mechanisms involved in pile-soil-structure interaction in liquefying soil, the data recorded during the 1995 Hyogoken-Nambu earthquake, shake table tests (e.g. Ohtomo; 1996, Yasuda *et al*.; 2000 and Mizuno *et al*.; 2000) and centrifuge tests (e.g. Dobry *et al*.; 1995, Abdoun *et al*.; 1997, Wilson *et al*.; 1999 and Wilson *et al*.; 2000) provide an insight into the mechanism of pile-soil-structure interaction in these circumstances.

Numerical procedures for the analysis of piles in liquefying ground have large uncertainties due to lack of understanding of the mechanisms involved in soil-pile interaction in the liquefying soil. Although numerical models based on two-dimensional and three-dimensional finite element analyses (e.g. Hamada *et al*.; 1994, Zheng *et al*.; 1996, Shahrour and Ousta; 1998 and Finn *et al*.; 2001) provide some insight into this interaction, they are computationally complex and time-consuming. Therefore in recent years, one-dimensional Winkler models based on finite element and finite difference methods for the seismic analysis of pile foundations have become popular amongst designers. In Winkler models, the pile is modelled as a beam and the lateral soil pressure acting on the pile is modelled using a non-linear spring and dashpot model. These methods are computationally very efficient and give results in a very short time.

In Winkler models, a *p-y* curve is used to define the behaviour of the nonlinear spring at any depth, where *p* is the soil resistance per unit length of the pile and *y* is the pile lateral displacement. These *p-y* curves should be back-figured from the field or model tests. However, for liquefying soil, available case histories and experimental data are limited. Therefore, this paper presents a method based on Mindlin's equation to determine the non-linear spring constants of the Winkler model. Depending on the amount of pore pressure development, spring coefficients in the spring-dashpot model are degraded. The effect of radiation damping is taken into account seperately. This is an extension of the method developed by Poulos (1982) for piles subjected to lateral soil movements under static conditions.

2 NUMERICAL MODEL

The numerical model developed for the analysis of piles in liquefying ground is based on the finite element method and involves two stages. First, an effective-stress-based free-field ground response analysis is carried out to determine the external soil movement and the degradation of soil stiffness and strength due to pore pressure generation. Then, a seismic analysis of the pile is carried out by applying the computed ground displacements dynamically to the pile. For the pile analysis, the soil shear modulus and ultimate lateral strength of the soil are calculated based on the effective stress level within the soil.

2.1 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. Input motions are applied at the boundary between the soil deposit and the bedrock through a viscous dashpot with damping, $r_{BR}V_{BR}$, where r_{BR} and V_{BR} are the density and shear wave velocity of the bedrock material respectively. This viscous damping is included in the analysis to take into account the effect of energy loss due to the dispersion of wave energy (Joyner and Chen, 1975). Pore pressure generation is calculated based on the effective stress method developed by Liyanapathirana and Poulos (2001). During the analysis, soil stiffness and strength are degraded, based on the effective stress level in the soil. At each time step, pile response is computed using the soil movements along the depth of the pile and the degraded soil stiffness obtained from the ground response analysis.

2.2 Pile Analysis

In the dynamic analysis of piles, moving soil interacts with the pile. In the vicinity of the pile, soil displacement is different from the displacement of the soil if there were no piles. Therefore, in the pile analysis, it is assumed that the displacement of the soil away from the pile can be represented by the displacements obtained from the free-field ground response analysis. Soil-pile interaction is modelled using the analysis method for a dynamically loaded beam on a non-linear Winkler foundation, where 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 Wunkler model. A solution to the problem can be obtained by solving Equation (1) using the finite element method.

The spring coefficients of the Winkler model, which represent the interaction between the pile and soil, are obtained by integrating the Mindlin's equation over a rectangular area (Douglas and Davis, 1964). Due to the gradual increase in pore pressure level of the soil, the shear modulus of the soil at any depth of the soil deposit changes with time. Therefore, these spring coefficients should be calculated at the beginning of each time step during the analysis.

The Mindlin hypothesis does not automatically satisfy the soil radiation damping and this should be incorporated into the analysis separately. Here the value of $5r_sV_s$ proposed by Kaynia in 1988 (Tabesh and Poulos, 2000) is used for the dashpot coefficient where r_s is the density of the soil and V_s is the shear wave velocity of the soil. This dashpot takes into account the radiation damping of the shear waves travelling away from the pile.

Time integration of the Equation (1) is performed using the constant average acceleration method. At each time step, the lateral pressure at the soil-pile interface is monitored and an iterative procedure is used to keep it at or below the ultimate lateral presssure at the pile-soil interface. When the lateral pressure at the pile-soil interface reaches the ultimate value, soil yielding occurs. For piles in sand, Broms (1964) has suggested that the ultimate lateral pressure can be given by,

$$P_y = N_p . P_p \qquad (2)$$

where N_p is a factor range between 3 and 5, and P_p is the Rankine passive pressure. The amount of radiation damping during soil yielding is still not known but several researchers have shown that it is far less than the value obtained from the elastic assumption. Nogami (1987) compared numerical results obtained from his model with field tests and concluded that neglecting the gaps due to soil yielding at the pile-soil interface results an overestimation of damping and an underetimation of pile deflection. Using centrifuge tests, Chako (1995) showed that the elastic formulation of radiation damping is valid only during small amplitude shaking and is excessive when large displacements due to soil yielding occurs. Therefore in the present analysis, the effect of radiation damping is neglected during soil yielding.

3 COMPARISON WITH CENTRIFUGE DATA

For the method described in the previous section, a computer program based on the C language has been developed and the ability of the method to simulate pile behaviour in liquefying soil has been demonstrated by simulating the centrifuge test performed by Wilson *et al*. (1999). The soil profile used for the centrifuge test consisted of two horizontal layers of saturated, fine, uniformly graded Nevada sand. The 11.4 m thick dense lower layer had a relative density (D_r) of 80% and the 9.1 m thick medium dense upper layer had a relative density of 55%. The structural model consisted of a single pile-supported structure and was equivalent to a steel pipe pile with a diameter of 0.67 m and a wall thickness of 19 mm. The pile extended 3.8 m above the ground surface and carried a superstructure load of 480 kN. The depth of pile embedment was about 15 m.

Table 1. Material properties for Nevada sand

Property	**Nevada Sand**	
	D_r **= 55 %**	D_r **= 80 %**
Mass density (kg/m^3)	2670.0	2670.0
Porosity	0.406	0.373
Low-strain shear modulus, G_0 (MPa)	28.0	41.46
Reference mean effective normal stress (kPa)	100.0	100.0
Friction angle, f'	34.15^o	39.5^o
Permeability (m/s)	6.05 x 10^{-5}	3.7 x 10^{-5}

Properties of the Nevada sand with relative densities 55% and 80% are given in Table 1. The variation of shear modulus along the depth of the soil deposit is given as (Popescu and Prevost, 1993),

$$G_s = G_o \left[\frac{\frac{1+2K_o}{3}\sigma'_v}{p_o} \right]^{0.7} \text{ MPa} \tag{3}$$

where K_o is the coefficient of lateral earth pressure at rest, p_o is the reference normal stress which is 100 kPa for sand (Popescu and Prevost, 1993) and G_o is the low strain shear modulus of the soil.

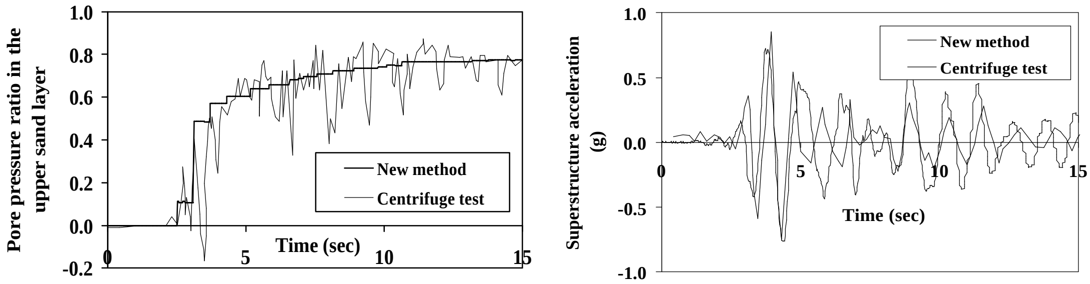


Figure 2. Comparison of time histories of pore pressure in upper sand layer and superstructure acceleration with centrifuge test by Wilson *et al*. (1999)

This model was subjected to the Kobe 1995 earthquake scaled to a maximum acceleration level of 0.22g with slight modifications to the frequency content. Figure 2 shows the measured and computed pore pressure distribution close to the surface of the soil deposit and the superstructure acceleration. About 3.5 sec after the application of earthquake loading, a sharp rise is observed in the pore pressure distribution and softening of the soil caused by this pore pressure increase can be seen in the acceleration time history as an increase in the fundamental period of the structure. Figure 3 shows the bending moment and displacement of the pile obtained at 11 sec after application of the earthquake loading. Despite the relative simplicity of the new method, the results obtained agree well with the values recorded during the centrifuge test. The reason for the slight difference observed in test results and numerical results may be due to the difference between frequency content of the Kobe earthquake record used for the analysis and the Kobe earthquake record generated for the centrifuge test.

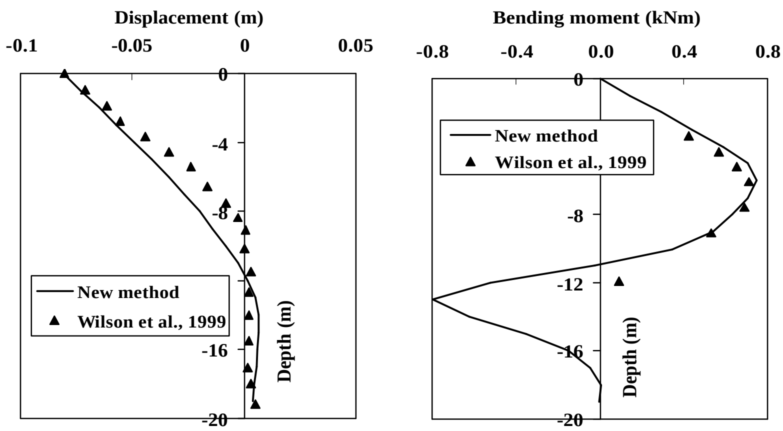


Figure 3. Comparison of Displacement and Bending moment along the pile 11 sec after application of the earthquake loading with centrifuge test by Wilson *et al*. (1999).

4 PARAMETRIC STUDY

In order to study the effects of soil liquefaction on pile performance, a parametric study has been carried out by varying properties of the pile, soil and the nature of the earthquake. For all analyses presented, it is assumed that the soil follows that Mohr-Coulomb failure criterion and the shear modulus of the soil is calculated using Equation (3) with a power of 0.5 instead of 0.7 because for many soil types, G_s is proportional to $\sqrt{\sigma'_v}$.

Results are presented for relative densities of 50%, 60%, 70%, 80% and 90%. For these relative densities, G_o in Equation (3) has been changed linearly from 30 MPa to 50 MPa and the effective friction angle of the soil has been changed linearly from 30^o to 45^o. In all cases, the length of the pile is set equal to the depth of the soil deposit which overlays the bedrock. The soil has a density of 1900 kg/m^3 and a permeability of 10^{-4} m/s. It is assumed that the bedrock has a shear modulus of 3500 MPa and a density of 2445 kg/m^3.

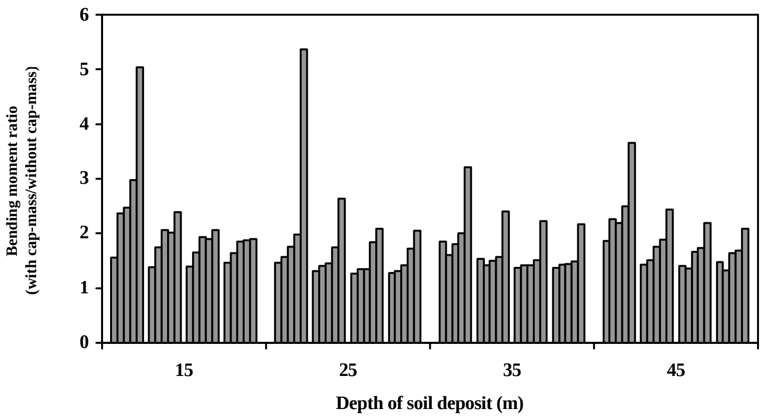
4.1 **Effect of cap-mass on pile performance**

The effect of cap-mass on piles founded in liquefying soil has been studied by comparing results obtained from a pile carrying a cap-mass and a pile with a rigid massless cap. Generally the superstructures supported by pile foundations are multi degree of freedom systems, but in the design of pile foundations, the superstructure is almost always reduced to a single mass at the pile head to simplify the analysis. It can change the response of a pile dramatically.

The cap-mass carried by a pile is determined based on the ultimate load carrying capacity of piles in sand. The input acceleration record used for the analysis is the 1995 Kobe earthquake record scaled to 0.15g. The analysis has been repeated for pile lengths of 15 m, 25 m, 35 m and 45 m and pile diameters of 0.3 m, 0.6 m, 0.9 m and 1.2 m.

Figure 4 compares the maximum bending moment obtained for piles with and without cap-mass. It can be seen that in all cases, the inclusion of cap-mass has increased the maximum bending moment developed in the pile, in some cases more than five-fold. The increase in relative density reduces the degree of soil liquefaction. This causes an increase in the maximum acceleration at the ground surface and hence the inertia force at the pile head when the pile carries a cap-mass. Also the cap-mass carried by a particular pile increases with the increasing relative density of the soil, because the cap-mass carried by a particular pile is calculated based on the ultimate load carrying capacity of the pile. Therefore, the significance of cap-mass increases with the increasing relative density of the soil for a particular pile. On the other hand, with the increase in degree of soil liquefaction, the influence of the inertia force at the pile head, on the pile performance decreases.

Although an increase in pile diameter causes an increase in maximum bending moment developed in a pile with or without cap-mass, the maximum bending moment ratio given in Figure 4 decreases with the increase in pile diameter. Therefore it can be concluded that the influence of inertia force at the pile head on the bending moment generated in the pile becomes less significant with increasing pile diameter.

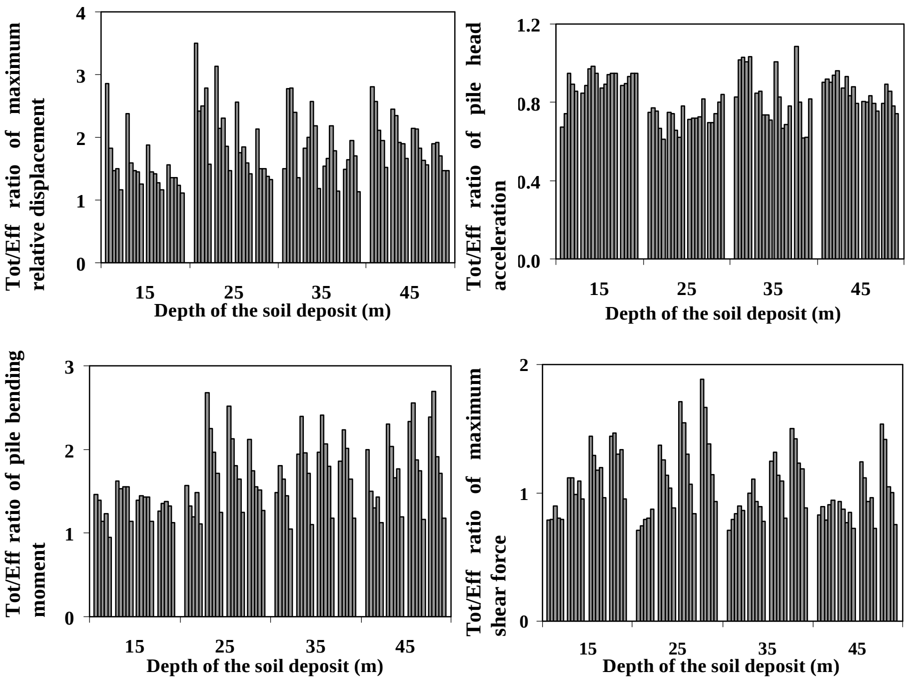


For each depth of soil deposit, first 5 columns are for d = 0.3 m (Dr = 50 %, 60 %, 70 %, 80 % and 90 %), second 5 columns are for d = 0.6 m, then d = 0.9 m and finally d = 1.2 m

Figure 4. Comparison of maximum pile bending moment with and without cap-mass.

4.2 Effect of pore pressure generation on pile performance

Although numerical methods are developed to obtain pile performance under earthquake loading, in many of them, the effect of soil stiffness and strength degradation due to pore pressure generation and subsequent soil liquefaction has not been incorporated. Therefore, in many instances, pile design is based on the maximum bending moments and shear forces calculated assuming soil as a linear elastic material or non-linear material neglecting stiffness and strength degradation due to cyclic loading. Therefore in this section, results obtained from the effective stress analysis described in Section 2 have been compared with a total stress analysis neglecting any pore pressure effects. In the total stress analysis, the soil is also assumed to follow the hyperbolic stress-strain relationship.



For each depth of soil deposit, first 5 columns are for d = 0.3 m (Dr = 50 %, 60 %, 70 %, 80 % and 90 %), second 5 columns are for d = 0.6 m, then d = 0.9 m and finally d = 1.2 m

Figure 5. Comparison of results obtained from the effective stress and total stress analyses.

Figure 5 shows the ratio between effective and total stress analyses for maximum pile bending moment, relative displacement, shear force and acceleration. It can be seen that the relative displacement and bending moment developed, shown in Figures 5 (a) and (c) respectively, increase when the pore pressure effects are included in the analysis. In many cases, the pile head acceleration shown in Figure 5 (b) has decreased when the pore pressure effects are included. This happens due to softening of the soil i.e. softening of the soil causes substantial pure bending in the pile and it smoothes high frequency peaks and troughs in the pile head acceleration.

A close examination of Figure 5 (d) shows that in some cases the inclusion of pore pressure effects into the analysis increases the maximum shear force developed in the pile but in others, it lessens the maximum shear force. If the maximum shear force is developed at the pile head, the inertia force at the pile head (cap-mass x pile head acceleration) controls the shear force. Therefore, the maximum shear force is decreased when the pore pressure effects are included. For the other cases, softening of the soil due to pore pressure effects has a more significant influence than the inertia force at the pile head.

4.3 Effect of the nature of the earthquake

In order to study the effect of the nature of the earthquake, earthquake records of Kobe-1995, Taft, Loma-Prieta, Pasadena, El-Centro, Niigata, Whittier Narrows, Melendy Ranche, Meckering, Cadoux, Superstition Hill, Northridge, Newcastle-89, Newcastle-94, Oolong, Tenant Creek, Gunjung, Saitama, New Zealand-73, Iran, San Fernando, Tomako, Miyagi, Tangshan and Elmore Ranche scaled to 0.1g, 0.15g and 0.2g, have been analysed. Each earthquake record has been applied to a 25 m long pile with 0.9 m diameter founded in a soil with a uniform relative density of 50%.

Figure 6 shows the variation of bending moment ratio obtained from an effective stress (including pore pressure effects) and a total stress analysis (neglecting pore pressure effects) with the 'Pseudo velocity' (Liyanapathirana and Poulos, 2001), *V*, which is the absolute area under the acceleration vs time curve. When *V* is greater than 2 m/sec, the degradation of soil stiffness and strength due to pore pressure generation has a significant influence on the pile performance. According to Figure 6, soil liquefaction can increase the bending moment developed in the pile by about four times compared to that given by a non-linear total stress analysis neglecting any pore pressure effects. However, this ratio depends on the nature of the earthquake. When *V* is less than 2 m/sec, the maximum bending moment obtained from total and effective stress analyses are nearly the same. The largest value of maximum bending moment ratio occurs when *V* is about 5 m/sec and beyond that, the bending moment ratio starts to decrease. With the increase in V, softening of the soil due to yielding occurs and an increase in bending moment is observed in the total stress analysis. As a result, the maximum bending moment ratio has decreased for cases with large *V*.

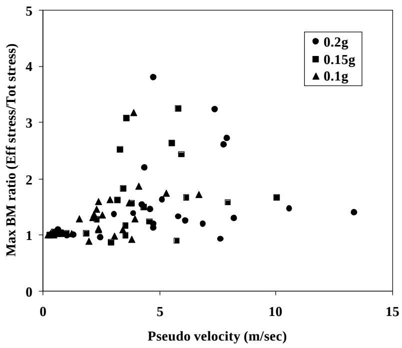


Figure 6. Variation of maximum bending moment ratio obtained from effective and total stress analyses with Pseudo velocity, *V*.

5 CONCLUSIONS

A numerical procedure has been described which can be used to simulate pile performance in liquefying soil. The ground displacements and degraded soil stiffness obtained from an effective stress based ground response analysis are applied to the pile dynamically to obtain the pile performance in liquefying soil. The spring coefficients of the Winkler model are derived from the Mindlin's equations. A centrifuge test has been simulated using the new method and the close agreement between results given by the numerical model and the data recorded during the centrifuge test demonstrates the ability of the new method to simulate pile behaviour in liquefying soil.

Results are also presented from a parametric study, varying geometric and material properties of the pile and soil, intensity of the earthquake and the nature of the earthquake. Results obtained for piles with a rigid massless cap and a cap-mass show that the inclusion of a cap-mass into the analysis has significantly increased the bending moment developed in the pile. The inclusion of pore pressure effects increases the bending moments and relative displacements of the pile but it reduces the maximum cap-mass acceleration. In some cases peak shear force developed in the pile increases due to inclusion of the pore pressure effects. In others the inclusion of pore pressure reduces the peak shear force developed in the pile due to the dependency of shear force on the cap-mass acceleration. It is found that the nature of the earthquake has a significant influence on the bending moment generated in a pile. If the pseudo velocity of the input acceleration record is greater than 2 m/sec, the maximum bending moment given by an effective stress analysis can be as high as 4 times that given by the total stress analysis. If the pseudo velocity is less than 2 m/sec, a total stress analysis can be used to obtain the pile behaviour.

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