2003 Pacific Conference on Earthquake Engineering

A pseudostatic approach for seismic analysis of piles in liquefying soil

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ABSTRACT: This paper presents a pseudostatic approach for the analysis of piles in liquefying soil, including the contribution of the superstructure to the pile and the interaction between the pile and the soil. The method involves two main steps. First an effective stress based ground response analysis is carried out to obtain the maximum ground displacements along the pile and the degraded soil modulus over the depth of the soil deposit. Next a static load analysis is carried out for the pile, subjected to the maximum free-field ground displacements and the static loading at the pile head based on the maximum ground surface acceleration. The method has been verified using an independent dynamic pile analysis program developed by the authors for the seismic analysis of piles in liquefying soil. The new method is then used to compute the response of pile foundations during Kobe 1995 earthquake and some centrifuge tests found in the literature where extensive soil liquefaction has been observed. Very good agreement is observed between the computed and the recorded pile bending moments.

1 INTRODUCTION

Liquefaction of saturated soil subjected to earthquake loading is one of the major factors affecting the behaviour of pile foundations and subsequent building failure in seismically active areas. The loss of soil stiffness and strength associated with progressive build up of pore water pressures in the saturated soil may lead to the development of large bending moments and shear forces in piles, thus resulting pile damage. The significance of pile damage due to soil liquefaction 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 a better insight into the pile-soil interaction during soil liquefaction can be obtained using three or two dimensional dynamic finite element models, they are often complex to apply and computationally expensive. Therefore, in recent years, one-dimensional Winkler models based on the finite element and finite difference methods for the seismic analysis of pile foundations have become popular, but many of them can be used only for the linear analysis of pile-soil interaction in non-liquefying soil (e.g. Novak; 1974, Kaynia and Kausel; 1982, Dobry *et al*.; 1982 and Kavvadas and Gazetas; 1993). For the non-linear analysis of pile-soil interaction in non-liquefying soil, numerical models have been developed by Penzien (1970), Kagawa (1980), Kagawa and Kraft (1981), Norris (1994), El Naggar and Novak (1996), Nogami and Konagai (1988) and Tabesh and Poulos (2001) but these models cannot predict the large bending moments and shear forces generated in piles due to soil liquefaction. For the seismic analysis of piles in liquefying soil, Winkler type models have been developed by Kagawa (1992), Yao and Nogami (1994), Fujii *et al*. (1998) and Liyanapathirana and Poulos (2002a).

Kagawa (1992) showed the significance of soil liquefaction on pile performance by comparing maximum bending moments obtained from piles in liquefying and non-liquefying soils. According to his study, when soil around the pile liquefies, the maximum bending moment developed in a pile can be as high as six times that occurring when the pile is in non-liquefying soil. Many pile failures observed during earthquakes are due to the inability of the pile to sustain such large bending moments.

Recently, pseudostatic approaches for the seismic analysis of pile foundations have emerged. In pseudostatic approaches, a static analysis is carried out to obtain the maximum bending moment and shear force developed in the pile due to earthquake loading. These methods are attractive for design engineers when compared to the difficult but more complete dynamic analyses. For piles in non-liquefying soil, Abghari and Chai (1995) and Tabesh and Poulos (2001) have developed pseudostatic approaches. When liquefaction is of concern, the stiffness of the soil is dramatically reduced and the effect of the reduced stiffness should be incorporated into the analysis. Therefore in this paper, a pseudostatic approach, which requires relatively little computational effort, is presented for the analysis of piles in liquefying soil. Results obtained from the pseudostatic approach are compared with the results given by a dynamic benchmark analysis developed by the authors and centrifuge data found in the literature. Despite its relative simplicity, the pseudostatic approach results are in good agreement with the observed pile behaviour during experiments and the dynamic benchmark analysis.

2 NUMERICAL PROCEDURE FOR BENCHMARK SOLUTION

The numerical model developed for the benchmark solution is based on the finite element method and involves two stages, as follows:

1. 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 (Liyanapathirana and Poulos; 2002b).
2. Dynamic analysis of the pile is carried out by modelling the pile as a beam, while the soil-pile interaction is modelled using the method of a dynamic beam on a non-linear Winkler foundation.

The spring coefficients of the Winkler model have been obtained by integrating Mindlin’s equation over rectangular planes representing the pile elements (Douglas and Davis, 1964). The degradation of soil modulus gives rise to non-homogeneity of the soil profile. The use of the Mindlin’s equation is of course approximate for soils which are not homogeneous and isotropic, but can give results of adequate accuracy for many cases of non-uniform soil profiles (Poulos, 1982). The dashpot, which takes into account soil radiation damping, is incorporated into the Winkler model separately. Here, the value of $5r_sV_s$ proposed by Kaynia in 1988 (Tabesh and Poulos, 2000) has been used where r_s and V_s are density and shear wave velocity of the soil respectively. It takes into account the attenuation of the shear waves travelling away from the pile. Coefficients of the Winkler model, which represents the interaction between the pile and the soil away from the pile, should be computed based on the degraded shear modulus of the soil during each time step of the analysis. Details of the benchmark analysis and validation of the method has been given by Liyanapathirana and Poulos (2002a, 2003).

3 PSEUDOSTATIC APPROACH

In 1995, Abghari and Chai presented a pseudostatic approach for the analysis of piles subjected to earthquake loading in non-liquefying soils. The inertial force acting at the pile head is represented by the product of cap-mass and spectral acceleration (Dowrick, 1977). By comparing the results given by the pseudostatic approach with the results given by a dynamic finite element analysis, they concluded that the inertial force should be reduced to 25% for the pile deflection and to 50% for the bending moment and shear force to obtain the results in agreement with the dynamic finite element analysis. They made this conclusion by analysing only one example and the method has not been generalised. However, they have taken into account the non-linear behaviour of the soil in their analysis.

Recently Tabesh and Poulos (2001) proposed that, by applying the full inertia force at the pile head, the pseudostatic approach provides results in agreement with the dynamic analysis. However, they carried out an elastic free-field site response analysis, and in the pile analysis the non-linear behaviour of the soil is also not taken into account. They observed an excellent agreement with the dynamic analysis for the cases without cap-mass, but when the cap-mass increased, they showed that the pseudostatic approach overestimates the maximum bending moment and shear force developed in the pile. Although Ishihara and Cubrinovski (1998) have developed a static approach for the pile foundations in soil deposits subjected to lateral spreading due to earthquake loading, the inertial force at the pile head has not been considered in computing the maximum bending moment and shear force developed in the pile.

Here the pseudostatic approach has been extended for a liquefying soil, where the degradation of shear modulus of the soil occurs with the generation of pore water pressure in the soil. Although spectral acceleration has been used by Abghari and Chai (1995) and Tabesh and Poulos (2001), it has been found that the inertial force at the pile head calculated using the spectral acceleration gives an overestimation of pile response, when the surrounding soil starts to liquefy. Hence, instead of spectral acceleration, the maximum acceleration at the ground surface has been used to calculate the inertia force at the pile head.

The calculation steps involved in this new approach can be summarised as below:

1. First, a free-field site response analysis is performed by taking into account the pore pressure generation and dissipation in the soil deposit due to the earthquake loading (Liyanapathirana and Poulos, 2002b). From this analysis, the maximum ground surface acceleration, maximum ground displacement along the length of the pile, the minimum shear modulus and effective stress level attained during the seismic activity can be obtained.
2. The superstructure is modelled as a concentrated mass at the pile head. Generally the superstructures supported by pile foundations are multi-degree-of-freedom systems but in the design of pile foundations, the superstructure is reduced to a single mass at the pile head to simplify the analysis.
3. The lateral force to be applied at the pile head is the cap-mass multiplied by the maximum ground surface acceleration obtained from the ground response analysis.
4. The pile-soil interaction is modelled using the spring coefficients calculated by integrating the Mindlin's equation as discussed before, based on the minimum shear modulus of the soil deposit at each depth, at any time, given by the free-field site response analysis (Step 1).
5. A non-linear static load analysis is carried out to obtain the profile of maximum pile displacement, bending moment and shear force along the length of the pile by applying the lateral forces calculated in Steps 3 and 4, and the soil movement profile calculated in Step 1, simultaneously to the pile.

4 VERIFICATION OF THE PROPOSED METHOD

The proposed pseudostatic approach has been verified using the dynamic benchmark analysis described before. In turn, the benchmark analysis has been verified through comparisons with field data and centrifuge data (Liyanapathirana and Poulos, 2002a). Results have been obtained by changing the length and diameter of the pile. It is assumed that the pile extends to the bottom of the soil deposit.

The shear modulus of the soil is assumed to vary with the effective stress level of the soil as shown below:

$$G_s = G_0 \left(\frac{\frac{1+2K_0}{3} s'_v}{100} \right)^{0.5} MPa \qquad (1)$$

where s'_v is the effective stress level of the soil, K_0 is the coefficient of earth pressure at rest and G_0 is a constant which varies with the relative density, D_r, of the soil.

Table 1. Maximum bending moment (MNm) obtained from dynamic and pseudostatic analyses (D_r= 50%)

Length	*d* = 0.3 m		*d* = 0.6 m		*d* = 0.9 m		*d* = 1.2 m	
	Dynamic	Static	Dynamic	Static	Dynamic	Static	Dynamic	Static
15 m	0.13	0.12	0.86	0.93	4.11	4.39	12.1	13.4
20 m	0.07	0.06	0.46	0.46	2.16	2.16	6.08	6.08
25 m	0.10	0.95	0.57	0.57	2.30	2.55	7.0	7.27
30 m	0.14	0.14	0.53	0.69	2.50	2.70	7.41	7.41

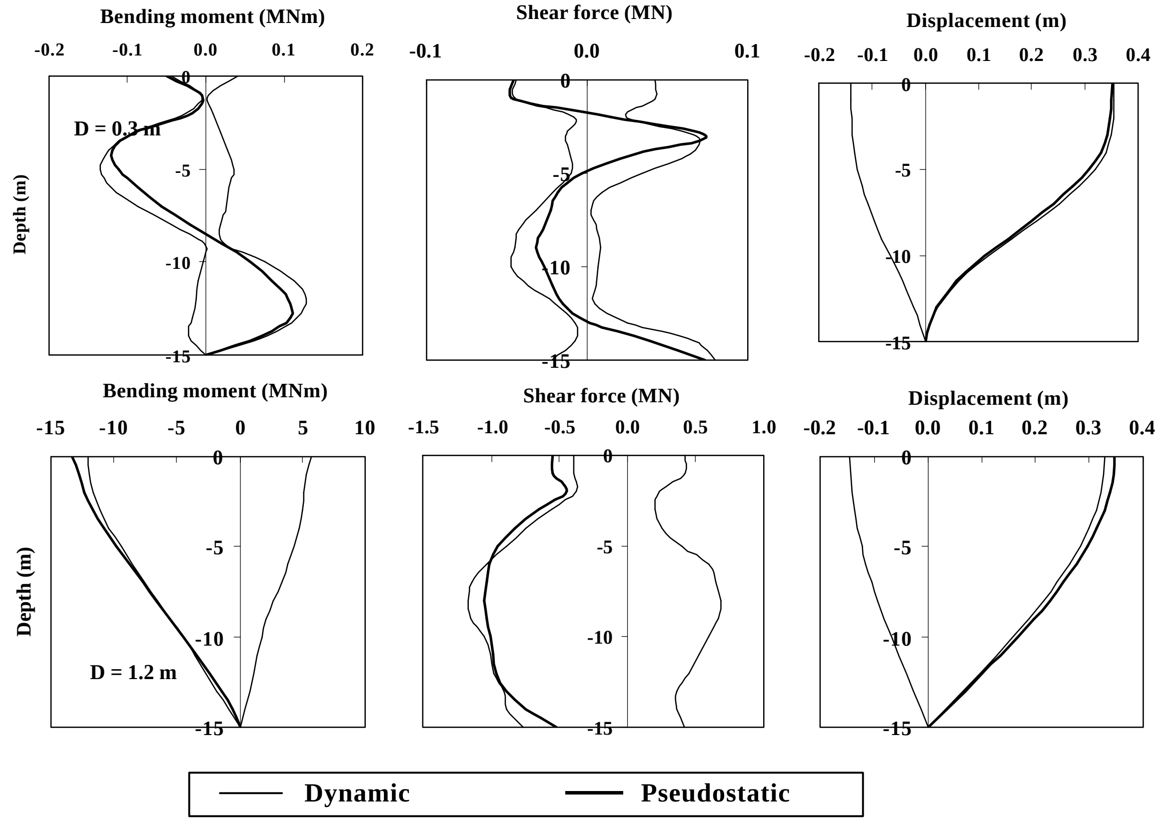


Figure 1. Variation of maximum pile bending moment, shear force and diaplacement along the depth for a 15 m long pile with diameters 0.3 m and 1.2 m (D_r = 50%).

Table 1 shows the maximum pile bending moment obtained for different pile configurations using the dynamic benchmark (Dynamic) and pseudostatic analyses (static) when the relative density of the soil is 50%. The depth of the soil deposit ranges between 15 m and 30 m. The soil deposit used for the analysis has a mass density of 1900 kg/m^3, permeability of 5.5e^{-5} m/s, friction angle of 30^0 and G_0 of 30 MPa. It is assumed that the water table is at 2.0 m below the ground surface. The liquefied depth for the four soil deposits considered for the analysis varied between 8 m and 6 m respectively for the 15 m and 30 m soil deposits.

For all cases, pile head is assumed to be fixed and the pile tip is considered to be restrained against lateral movement. The 1995 Kobe earthquake record scaled to 0.25g has been used as the excitation source. The cap-mass carried by each pile configuration is calculated based on the ultimate load carrying capacity of piles in sand, with a factor of safety of 2.5. Despite its simplicity, Table 1 shows that the pseudostatic analysis gives results in close agreement with the benchmark dynamic analysis.

The agreement between results is not only confined to the point of maximum bending moment but occurs along the whole length of the pile. Figure 1 shows the maximum positive and negative bending moment, shear force and displacement along the pile, over the duration of the earthquake, obtained from the dynamic analysis, for the 15 m pile with 0.3 m and 1.2 m diameters given in Table 1. These figures also show the maximum bending moment, shear force and displacement obtained from the pseudostatic analysis. There is a close agreement between the dynamic and static analyses along the length of the pile. It is interesting to see that in some parts, the static profile matches with the maximum positive envelope and in other parts, it matches with the maximum negative envelope.

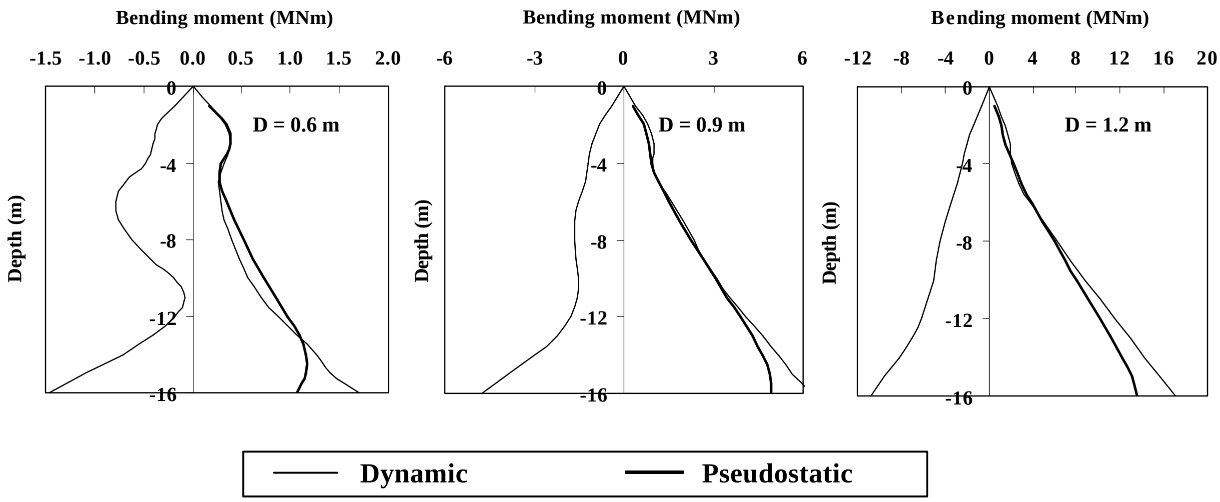


Figure 2. Variation of pile bending moment along depth for a free head pile with diameters 0.6 m, 0.9 m and 1.2 m (D_r = 50%)

In Figure 2, the maximum bending moment and shear force profiles along the 15 m long pile with D_r = 50 % are given for a free head pile where the pile extends 1 m above the ground surface. Maximum bending moment profiles are given for pile dimeters of 0.3 m, 0.6 m, 0.9 m and 1.2 m. When carrying out the static analysis, the pile head is assumed to be at the ground surface. Hence in addition to the inertial force, the moment due to eccentricity of the inertial force acting at the pile head is applied to the pile in the static analysis. The agreement between dynamic and static analyses confirms that, irrespective of the boundary conditions at the pile head and the pile tip, the pseudostatic approach can be used to obtain the internal response of the pile.

Although results are given here only for some selected cases, the method has been found to give reasonable agreement with the dynamic analysis for a wide range of soil conditions and pile configurations. It has been found that the maximum values are given at the same depth and the difference in magnitude is less than 25%, which is generally acceptable for practical pile design purposes.

5 COMPARISON WITH FIELD AND CENTRIFUGE DATA

In this section, the proposed pseudostatic method has been used to estimate the maximum bending moments developed in a pile used for a centrifuge test carried out by Abdoun *et al*. (1997), and the bored piles at Bridge Pier 211 in Uozakihama Island after the Hyogoken-Nambu 1995 earthquake, reported by Ishihara and Cubrinovski (1998).

The centrifuge test by Abdoun *et al*. (1997) was carried out to study the pile response during lateral spreading. The model consists of three sand layers. The middle layer is a 6 m thick Nevada sand layer with relative density of 40 % and the shear modulus of this layer is given by Equation 1 with G_0 of 25 MPa. In the present numerical analysis, it was assumed that the middle sand layer has a density of 1800 kg/m^3, friction angle of 33^0 and permeability of 7x10^{-5} m/s. Two cemented sand layers at the top and bottom of the centrifuge model were 2.0 m thick and pervious. The input acceleration record generated for the test was a sine wave with amplitude of 0.25g and frequency of 2 Hz over a period of 20 seconds. In the numerical simulation, it was assumed that the pore water pressures are generated only in the middle Nevada sand layer. In the centrifuge test, the sine wave was applied to the pile-soil system, 11 seconds after the beginning of the test.

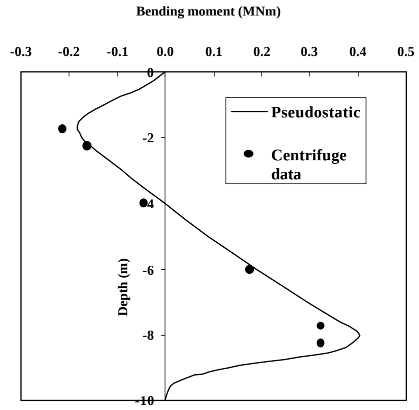
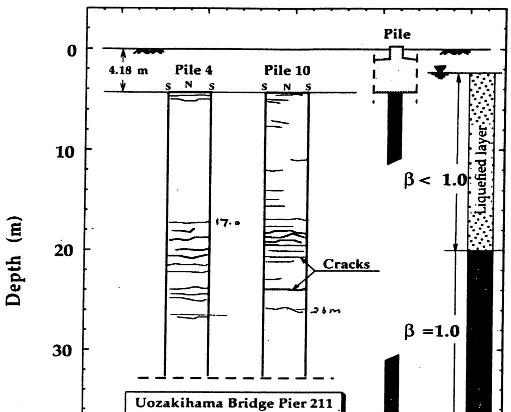


Figure 3. Comparison of maximum bending moment along the pile with centrifuge data (Abdoun et al., 1997)

Figure 3 shows the maximum bending moment profile obtained from the pseudostatic analysis and the maximum bending moments obtained at several depths during the centrifuge test. It can be seen that the calculated values agree well with the values recorded during the centrifuge test.

Next, the field measurements made in the piles at the Pier 211 in Uozakihama Island after the Hyogoken-Nambu earthquake occurred on 17th January 1995 reported by Ishihara and Cubrinovski (1998) has been simulated using the pseudostatic approach presented in this paper. Figure 4 (a) shows the crack distributions observed in piles after the earthquake. Piles at bridge Pier 211 are 46 m long and the diameter is 1.5 m. The water table is 2.0 m below the ground surface and the upper 20 m of this site consists of Masado sand with an initial shear modulus of 57.8 MN/m^2 and density of 2000 kg/m3 (Tokimatsu et al., 1998). Soil liquefaction was observed in the Masado sand layer below the water table only. Therefore only the top 20 m layer was analysed using the effective stress method incorporating pore pressure generation and dissipation. After liquefaction, the effective stress level in the soil was reduced to a minimum of 2% of the initial effective overburden pressure. For this analysis the cyclic shear strength curve for the Masado sand given by Ishihara (1997) was used. It was assumed that the base rock has a density of 2200 kg/m^3 and a shear modulus of 75 GN/m^2.



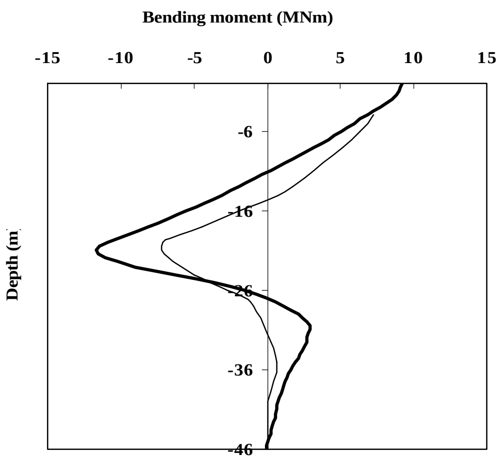


Figure 4. (a) Cracks observed in piles at the bridge pier 211 in Uozakihama Island (Ishihara and Cubrinowski, 1998) and (b) Bending moment of the pile at Bridge Pier 211 in Uozakihama Island calculated from the pseudostatic approach and Ishihara and Cubrinovski (1998) results (*b*=0.01).

Figure 4 (b) shows the maximum bending moment profile along the pile obtained from the pseudostatic approach and also that calculated by Ishihara and Cubrinovski (1997). The lower end of the RC pile is assumed to be fixed while the pile head is assumed to be fixed to the footing but free to move in the horizontal direction. The predictions made by the pseudostatic approach agree well with the results given by Ishihara and Cubrinovski (1998). The yield moment for these piles is about 5 MNm. The computed maximum bending moment profile exceeds the yield moment near the pile head and in the vicinity of the boundary between the liquefied and non-liquefied layers. This is consistent with the location of cracks observed after the earthquake shown in Figure 4 (a).

6 **CONCLUSIONS**

This paper has described a pseudostatic approach that can be used to compute maximum bending moment and shear force developed in a pile founded in liquefying soil. An effective stress based free-field ground response analysis is first carried out and the resulting ground displacements, degraded soil stiffness and inertial force at the pile head, based on the cap-mass and the maximum ground surface acceleration, are applied to the pile statically to obtain the internal pile response. The spring coefficients of the Winkler model used in the pseudostatic analysis are derived from Mindlin's equation. The results presented in the paper suggest that the new method has promise in practical applications. For a few cases the new method overestimated the pile bending moment and shear force but the values are generally within 25% of those obtained from the dynamic analysis. Both dynamic and pseudostatic analyses give peak values at the same locations. The pile performance observed during a centrifuge test and a real earthquake has been simulated using the pseudostatic approach. It is found that the pile response calculated from the new method is close to the observed behaviour.

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