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Kinematic Bending Moment of Piles under Seismic Motions

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ABSTRACT

Soil pile structure interaction under seismically loadings is a very complex phenomenon involving of kinematic and inertial interaction among soil, pile and superstructure. Piles experience kinematic bending moments caused by the soil deformations due to the passage of the seismic waves through the surrounding layered soil. These moments are increased at each the levels of the soil deposit with the different modulus. A model of the soil pile interaction 2D is developed in this paper. The main results of parametrical study on single piles obtained by this model are compared with the exiting design methods for evaluating the kinematic interaction between soil-pile subjected to the seismic excitations. The shear strain and the shear stress are evaluated by the linear equivalent method of the free field site response analyses. The results show that the kinematic bending moment at the interface is changed by the soil nonlinearity behavior and the frequency content of the seismic motion even in absence superstructure.

Key words: Soil-pile interaction, interface, kinematic bending moment, simplified methods, site response

INTRODUCTION

Significant damages of structures supported by deep foundations due to complete or partial collapse has been observed in the past earthquakes. The 1964 Niigata Earthquake, the 1964 Alaska Earthquake, The Loma Prieta Earthquake in 1989 and recently during the 1995 Kobe Earthquake have been demonstrated paramount importance of the soil-pile-superstructure interaction in the seismic behavior of structures (Meymand, 1988). It is apparent that the Soil-Pile-Interaction (SPI) has a substantial role in the structure design. As this complex phenomena and highly coupled has received considerable attention (Cairo and Dente, 2007; Castelli and Maugeri, 2009).

The Soil-pile-superstructure Interaction (SPSI) included two sources: Inertia and Kinematic Interaction. The kinematic interaction is due to the presence of pile foundation on or in the ground surface that causes the ground motions deviate from free-field notions. it is mentionable that the method of the dynamic site analysis and soil conditions effect the free-filed site response (Khari and Bazyar, 2008). Inertia interaction is due to the kinematic interaction transmitted to the superstructure (Castelli and Maugeri, 2009). While, the inertial loading effects are only considered in professional design offices but impotence of the kinematic Interaction has been recognized by some building codes such as Euro code 8 that states the kinematic information should be taken into account under several conditions (Kavvadas and Gazetas, 1993; Mylonakis, 2001).

Evaluation of the soil-pile interaction has developed by several investigators. Exiting methods can be classified into numerical approaches, Beam on Nonlinear Winkler Foundation method

(BNWF) and simplified formulations. In the numerical approaches-so called direct approaches-SPSI is completely modeled and the seismic response is assessed in one step. Maheshwan *et al.* (2004), Kimura and Zhang (2000) and Wu and Finn (1997) implemented the Finite Element Method (FEM) in their researches, whereas (Cairo and Dente, 2007; Kaynia and Kausel, 1982) used the Boundary Element Method (BEM)for the seismic behavior of pile foundations. Although, the above two methods are versatile technique as SSPI analysis can be performed coupled and independent from the site response analysis but they are very expensive from a calculation viewpoint. The Beam on Nonlinear Winkler Foundation (BNWF) method (studies of Nogami *et al.*, 1992; El-Naggar *et al.*, 2005; Maheshwari and Watanabe, 2006) and simplified formulations (researches of Dobry and O'Rourke, 1983; Mylonakis, 2001; Nikolaou *et al.*, 2001; Liyanapathirana and Poulos 2005; Castelli and Maugeri, 2009) are widely used in research practices. It is worthy of note the results of the two last methods are in a good agreement with the mathematical analyses. This study presents the results of the different types of the dynamic numerical analysis to determine of the kinematic bending moments at the interface between the two soil layers. The main objectives of this paper are: (1) To evaluate the influence of soil properties and the characteristic of seismic excitation on the kinematic interaction of piles and (2) To determine of the kinematic bending moments at interface using the simplified formulations.

BNWF METHOD

The Winkler' foundation assumptions are implemented in the Beam on Nonlinear Winkler Foundation (BNWF) method. The Soil-pile behavior can be modeled nonlinearity and linearity based on solutions envisaged in the time and the frequency domain. More further, the nonlinearity soil behavior, the radiation damping, the hysteretic damping and gapping have been carried out in BNWF models.

The pile is divided into a series of elastic finite beam-column segments and the mass of each element is lumped at nodal locations. The soil stiffness and damping is modeled by a series of springs (nonlinear or linear) and dashpots. The springs and dashpots are connected to nodal locations. There are several methods for calculation of the spring and the dashpot coefficients such as the p-y curves recommended by American Petroleum Institute (API) and the laboratory test method is commonly referred as the “pot-test”. The displacement time histories obtained of the free field site response analysis are entered to end of the springs. This method is widely used in research practices due to computationally time saving and accurate.

SIMPLIFIED APPROACHES

In the past two decades, the simplified methods have been developed due to little computational effort for the analysis of pile group or single piles. These methods are based on several assumptions such as the soil layer is linearly elastic and isotropic and the pile behavior is same semi-infinite beam. The main assumption of the simplified approach was suggested by Margason and Holloway (1977) that the pile follows the free-field soil motion. On the other hand, the singular disadvantage of the simplified method is to ignore the soil-pile interaction. Under these conditions, the bending moment at depth (z) can be computed by the following equation:

$$M(z,t) = E_p I_p \frac{1}{(z,t)} \tag{1}$$

where, 1/R(z,t) is the curvature of the vertical line, E_p and I_p are the pile elastic modulus and the pile moment of inertia, respectively. In practical case, the above equation is not suitable in the soil layered because the soil profile, in practical case, is rarely homogenous and the soil properties are different at depth. Several investigators have been developed a number of formulations for evaluation of the kinematic bending moment at the interface between two layers. The first procedure was developed by Dobry and O'Rourke (1983). They assumed each layer of the soil is homogenous and isotropic with the shear module G_1 and G_2. The shear strains are calculated with $\gamma_i = \tau/G_i$. The pile bending moment at the interface between two layers:

$$M = 1.86\left(E_p I_p\right)_{\frac{3}{4}}\left(G_1\right)\frac{1}{4}\gamma_1 F \tag{2}$$

where, F is a function of the ratio $C = (G_2/G_1)^{1/4}$:

$$F = \frac{\left(1 - C^4\right)\left(1 + C^3\right)}{\left(1 + C\right)\left(C^{-1} + 1 + C + C^2\right)} \tag{3}$$

The shear strain at the upper layer (γ_1) can be computed by the free-field site response analysis and also, it can be determined by the following equation (Seed and Idriss, 1982):

$$\gamma_1 = \frac{r_d \rho_1 H_1 a_{max,s}}{G_1} \tag{4}$$

where, $a_{max,s}$ the maximum acceleration at surface based on seismic zonation; H_1 and ρ_1 are the thickness and the density of the upper layer, respectively. r_d (=1-0.015z) is the depth factor; z is the depth from the ground surface (only z = 15 m).

Nikolaou *et al.* (2001) developed another simplified method based on the BNWF model. The kinematic pile bending moment is expressed by the Eq. 5:

$$M = 0.042\tau_c d^3\left(\frac{1}{d}\right)0.30\left(\frac{E_p}{E_1}\right)0.65\left(\frac{V_{s2}}{V_{s1}}\right)0.5 \tag{5}$$

where, V_{s1} and V_{s2} are the shear wave velocity in the upper and lower layer, respectively.

In Mylonakis (2001) presented the second method. Both of the radiation and the hysteretic damping were taken into account by him. Base on his studies, the maximum bending moment was expressed as:

$$M = \frac{\left(E_p I_p\right)\left(\frac{\varepsilon_p}{\gamma_1}\right)Q\gamma_1}{r} \tag{6}$$

while, r is the pile diameter; γ_1 is the strain of the upper layer; Q is an amplification factor so that its value is less than 1.25 (usually Q is equal to 1). $_p/\gamma_1$ is the strict strain transfer function which can be computed by the following equation:

$$\frac{\varepsilon_p}{\gamma_1} = \left(\frac{c^2 - c + 1}{2c^4}\right)\left(\frac{H_1}{d}\right)^{-1}\left\{\left[3\left(\frac{k_1}{E_p}\right)^{\frac{1}{4}}\left(\frac{H_1}{d}\right) - 1\right]c(c-1) - 1\right\} \tag{7}$$

where: $k_1 = \delta E_1$ and δ can be determined by the following equation:

$$\delta = \frac{3}{1-\vartheta^2}\left(\frac{E_p}{E_1}\right)^{-\frac{1}{8}}\left(\frac{L}{d}\right)^{\frac{1}{8}}\left(\frac{H_1}{H_2}\right)^{\frac{1}{12}}\left(\frac{G_1}{G_2}\right)^{-\frac{1}{30}} \tag{8}$$

where, ν is the Poisson's ratio; the free-field site analysis is suggested for estimate the peak shear strain γ_1 by Mylonakis (2001). In addition, Mylonakis stated that the peak shear strain can be computed by the Eq. 4. The maximum shear stress at the interface τ_c is:

$$\tau_c = \alpha_{max,s}\rho_1 H_1 \tag{9}$$

PARAMETRIC STUDY

To evaluate the kinematic bending moment at interface a single pile subjected to seismic excitations a 2D finite element model was presented by PLAXIS code. The results of this simulation were used to verify the results of the simplified approaches. The overall dimensions of the model boundaries included a width of 11D (D = pile diameter) and a high equal to the thickness of the two subsoil layers. The model was meshed by 15-node wedge elements. While, the horizontal outer boundary mesh of the model was fixed against displacements (Ux, Uy) but the vertical outer boundary, only was fixed in the horizontal displacement (Uy). As Fig. 1b shown, to absorb outing

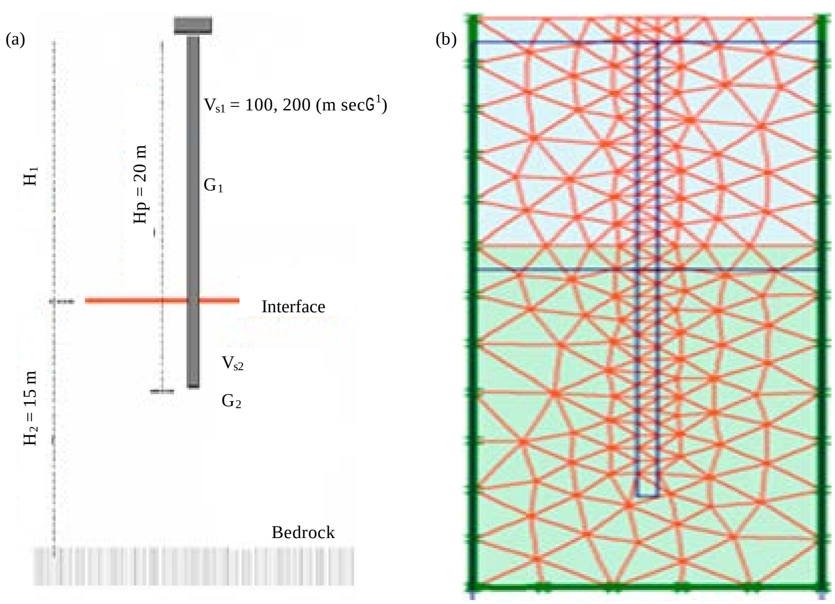

Fig. 1: Refrence scheme model (a) Soil model, (b) Typical 2D model for FE analysis

waves the absorbent boundary conditions were used in the outer boundaries. The surrounding soil and the single pile were considered as the Mohr-Coulomb non-associated floe rule and linear-elastic material, respectively. The soil-pile interaction was modeled by the interface element. The soil shear modulus reduction can be considered by reduction factor (R_{inter}) in PLAXIS. The kinematic interaction have been performed for a single pile with a length L = 20 (m); Young's modulus $E_p = 2.5\times10^7$ (kN m^{-3}); diameterd = 60 (cm); mass density ρ_p = 2.5 (Mg m^{-3}) and the Poisson's ratio $\nu = 0.15$.

As Fig. 1a shown, the pile is embedded in ideal two-layered subsoil at a depth H = 30 (m). The thickness of the second layer is assumed H_2 = 15 (m) while the thickness of the upper layer H_1 is variable (5, 10, 12, 15 and 18 m). The shear wave velocity of the upper layer is taken as 100 and 200 (m sec^{-1}), while V_{s2} is assumed the ratio equal to 2 and 4 for both of values of V_{s1}. Mass density and Poisson's ratio of the soil are: ρ_s = 1.97 (Mg m^{-3}) and ν = 0.4, respectively. The Young's modulus can be computed based on the shear modulus (E = 2G (1+ν)). In addition, the undrained shear strength was calculated based on the ratio suggested by the Applied Technology Council (G_{max}/S_u = 1000).

The average shear wave velocity can be computed by the following equation:

$$V_{s,30} = \frac{30}{\sum_{i=1,n} \frac{hi}{V_{s,i}}} \tag{10}$$

According to EN-1998-1, the soil profiles can be classified as type D, C and B. Acceleration time histories selected are scaled to the same peak ground acceleration of 0.1 g. Figure 2 shows the two acceleration time histories and spectral acceleration selected at the bedrock roof.

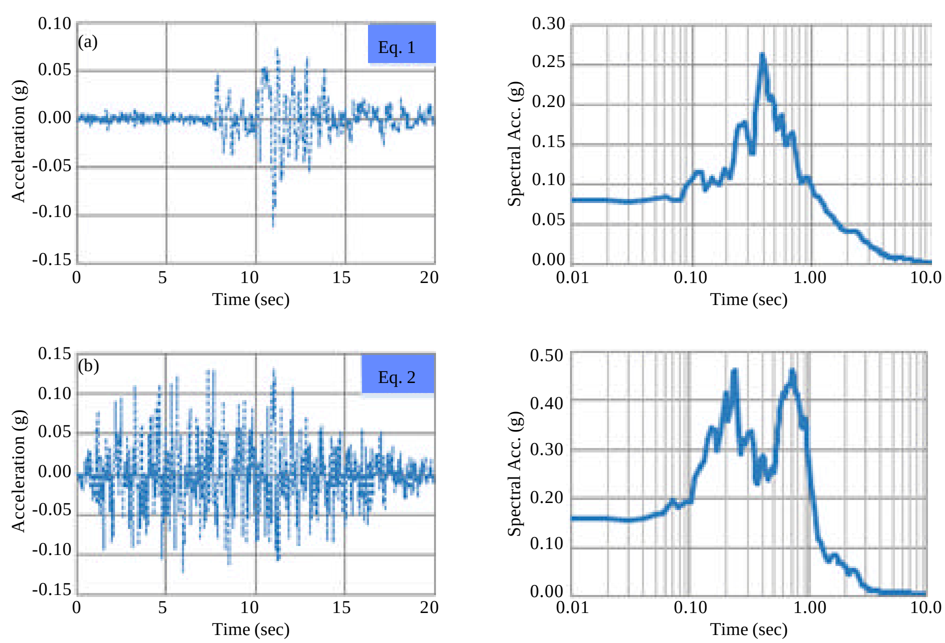

Fig. 2: Acceleration Time Histories and Spectra Response at the bedrock (a) Earthquake 1 and (b) Earthquake 2

ANALYSIS RESULTS COMPUTED WITH SIMPLIFIED METHODS

Kinematic bending moments at the interface of the two layers were computed by the simplified approaches developed by Dobry and O'Rourke (1983), Mylonakis (2001) and Nikolaou *et al.* (2001). The shear strain at the bottom of the first layer (γ_1) can be evaluated by Eq. 4 and the free field site response analysis. The free field site response was estimated by the Equivalent Linear Method (ELM). It is worthy of note that how model the soil dynamic parameters in the ELM affect the soil response (Khari *et al.*, 2011). While, the methods provided by Dobry and O'Rourke (1983) and Mylonakis (2001) are useable with the shear strain obtained by the above two methods but the simplified approach developed by Nikolaou *et al.* (2001) has been applied the shear stress determined with Eq. 9. The values of the shear stress calculated with Eq. 9 and the values directly computed by the equivalent linear method are shown at the Fig. 3 and 4. The value of the shear stress increase with increasing the thickness of the upper layer (H_1), whereas the shapes of the two figures are similar (Fig. 3). As Fig. 4 shows the shear strains are increased in the strong motion entered. as a result, the pile suffer the moments more based on the simple beam theory.

The kinematic bending moment at the interface the two-layered subsoil with the ratio of the shear wave velocities (Vs_2/Vs_1) equal to 2 and 4 are shown at the Fig. 5 and 6, respectively. The

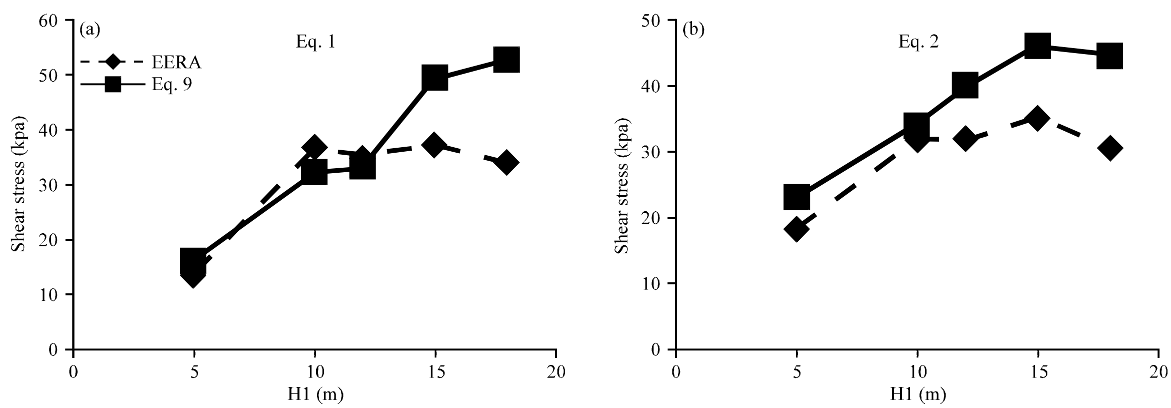

Fig. 3: Maximum shear stress at the interface between the two layers V_{s1} = 100 m sec^{-1}, (a) Earthquake 1 and (b) Earthquake 2

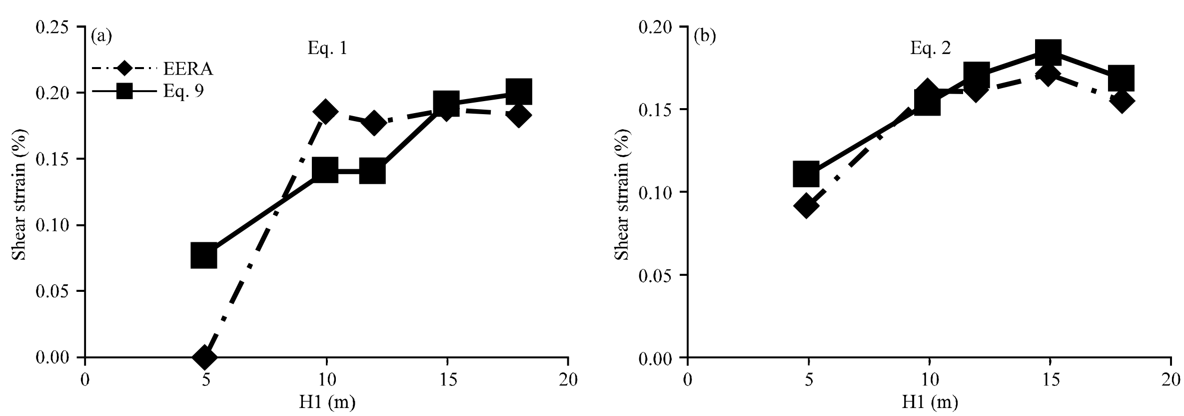

Fig. 4: Maximum shear stress at the interface between the two layers V_{s1} = 200 m sec^{-1}, (a) Earthquake 1 and (b) Earthquake 2

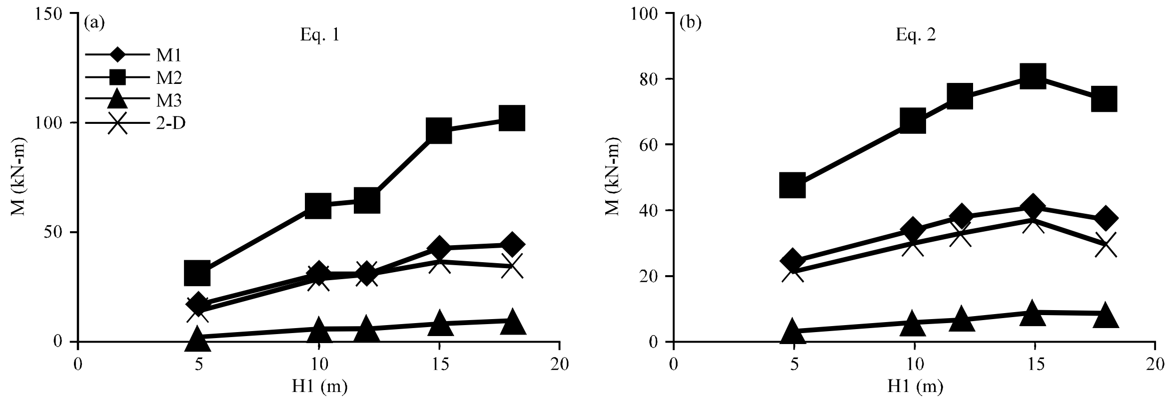

Fig. 5: Bending Moments predicted for V_{s1} = 100 (m sec^{-1}) and V_{s2}/V_{s1} = 2 Dobry and O'Rourke (1983) [M1]; Nikolaou *et al.* (2001) [M2], Mylonakis (2001) [M3] (a) Earthquake 1 and (b) Earthquake 2

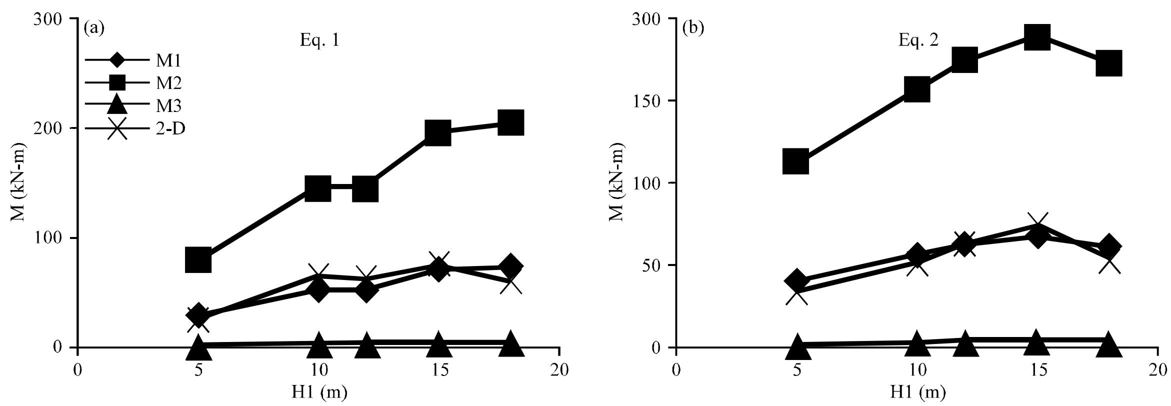

Fig. 6: Bending Moments predicted for V_{s1} = 100 m sec^{-1} and V_{s2}/V_{s1} = 2 Dobry and O'Rourke (1983) [M1]; Nikolaou *et al.* (2001) [M2]; Mylonakis (2001) [M3] (a) Earthquake 1 and (b) Earthquake 2

moments are increased with increasing the values of the first layer thickness. However, this increasing is different due to the input motion. The values predicted of the kinematic bending moments by the Nikolaou's method are highest. From Fig. 5 and 6, it emerges that the ratio of the shear wave velocity is effective on the values of the moment at the interface. As illustrated Fig. 6, the moments are increased with increased of the shear wave velocity at the second layer.

CONCLUSION

This study has been presented the results of the parametric study of the dynamic analysis of the kinematic bending moments of the single pile using the simplified methods in the two layers subsoil. The two acceleration time histories were selected as the input motion.The records were scaled to value equal to 0.1 g.

From the obtained results, the following conclusions may be drawn:

- The type of simplified method is strongly effective on the kinematic bending moments
- To close the fundamental period of the input seismic motion and the deposit soil, effects the maximum kinematic bending moments
- The values of the kinematic bending moment directly determined with the site response analysis are less than the values of the obtained by Eq. 9 especially when the thickness of the upper soil layer is increased

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