Evaluation of kinematic bending moments in a pile foundation using the finite element approach

F. Grassi & M. R. Massimino
Department of Civil and Environmental Engineering, University of Catania, Italy

Abstract

During earthquakes seismic waves crossing through soft soil can lead to significant curvatures on pile foundations, which in turn lead to significant bending moments. These bending moments are commonly named “kinematic bending moments” to be distinguished from the “inertial bending moments” due to horizontal forces transferred from superstructures to pile heads. Approaches to carefully evaluate inertial bending moments have been developed world-wide; on the contrary, up until now different simplified approaches have only led to different evaluations of the kinematic bending moments on pile foundations. Thus, the evaluation of kinematic bending moments is still questionable. Nevertheless, the European technical code EC8 and the new Italian technical code, D.M. 14/01/2008, underline the importance, in the geotechnical and structural design of pile foundations, of taking into account not only inertial bending moments, but also kinematic bending moments, in order to avoid significant structural damage.

In this paper a 3D soil-pile FEM system is analysed. The system is subjected to a seismic input motion, applied at the base of the system, which represents the conventional bedrock. The FEM analyses lead to the evaluation of the kinematic bending moment distribution along the pile. Finally, the numerical results are compared with those coming from some simplified approaches available in geotechnical literature.

Keywords: earthquake, pile foundation, kinematic bending moment, finite element method, Rayleigh damping.

WIT Transactions on The Built Environment, Vol 104, © 2009 WIT Press
www.witpress.com, ISSN 1743-3509 (on-line)
doi:10.2495/ERES090431

1 Introduction

The behaviour of pile foundations subject to seismic input motion depends on a complex interaction of three components: the soil, the pile and the superstructure. The different layers of foundation soil subjected to seismic waves coming from the bedrock drag in their motion piles. In this case, we talk about kinematic interaction. The bending moments caused by this kind of interaction are called kinematic moments.

In comparison with the free-filed condition, the presence of piles changes the seismic motion that involves the superstructure. The oscillation of the superstructure, prompted by the said seismic motion, causes the development of inertia forces, which in turn determine stresses and deformations in the foundation and soil, with the generation of additional waves at the soil-pile contact. In this case we talk about inertial interaction. The bending moments that are generated in the pile foundation due to the inertia forces coming from the superstructure are called inertial moments.

The relative importance of these two types of interaction (kinematic and inertial) depends on the characteristics of: the structure, the foundation, the foundation soil and the nature of the seismic waves (AGI [3]; Maiorano and Aversa [4]).

Until a few years ago, kinematic interaction was neglected and the seismic design of pile foundations and superstructures was based only on the inertial interaction. However, recent studies (Mylonakis [5]; Maiorano and Aversa [4]; Cairo and Dente [6]) have demonstrated the great importance in some cases of kinematic interaction, especially when a pile is embedded in two layers of soil with significantly different stiffnesses. Recently, the EC8 [1] and the new Italian Technical Regulations [2] prescribe to take into account both types of interactions for particular situations related to: the soil type, the seismicity of the area, and the importance of structure. For the sake of computational simplicity, it is preferable to separate the two interaction phenomena and to obtain the response of the soil-pile-superstructure system from the overlap of their single responses. This approach is commonly named "method of substructures" (Gazetas and Mylonakis [7]). The present paper is devoted to kinematic interaction.

The kinematic interaction has been studied with various models, such as: i) simplified models with the hypothesis that the pile follow the motion of soil in free-field condition (Margason [8]; Margason and Halloway [9]; NEHRP [10]); ii) Winkler models (BDWF), which summarizes the soil-pile interaction, with a system of springs and dampers distributed along the pile and a linear-elastic (Dobry and O'Rourke [11]; Nikolaou et al. [12]; Mylonakis [5]; Nikolaou et al. [13]; Sica et al. [14]; Castelli et al. [15]), or non-linear and hysteretic soil behaviour (Conte and Dente, [16, 17], Castelli and Maugeri [18]; Maiorano et al., [19]; Cairo et al. [20]); iii) FEM or BEM models (Wu and Finn [21], Maiorano and Aversa [4]). In particular, some Winkler models (BDWF) provide fairly simple formulas to be used for determining the maximum kinematic moment at the interface between two soil layers with different stiffnesses (Dobry and O'Rourke [11]; Nikolaou et al. [12]; Mylonakis [5]; Nikolaou et al. [13]).

In the present paper a 3-D FEM model is utilised. The numerical analyses are performed considering some schemes reported in Maiorano and Aversa [4]. The numerical results obtained by the Authors are compared with those obtained by Maiorano and Aversa [4], as well as with the results obtained by applying some of the previously mentioned Winkler models.

2 Utilised Winkler models for the comparison with FEM results

One of the most famous Winkler models to study pile foundation kinematic interaction is Dobry and O'Rourke's model [11]. On the basis of this model, the moment at the interface between two soil layers with different stiffnesses (fig. 1) can be estimated with the following expression:

$$M = 1.86(E_P I_P)^{3/4}(G_1)^{1/4}\gamma_1 F \tag{1}$$

where

$$F = \frac{(1-c^{-4})(1+c^3)}{(1+c)(c^{-1}+1+c+c^2)}; \quad C = \left(\frac{G_2}{G_1}\right)^{1/4}; \quad \gamma_1 = \frac{\tau}{G_1} \tag{2}$$

and also: E_P and I_P are the pile Young modulus and inertia moment; G_1 and G_2 are the shear moduli of the two layers of soil, τ is the maximum stress derived from local seismic response.

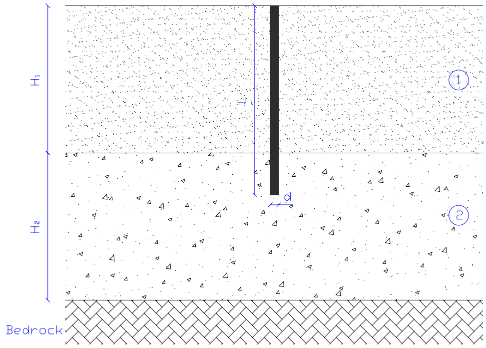


Figure 1: Reference scheme of a pile embedded into two soil layers of different stiffnesses.

Nikolaou et al. [12], have investigated the behaviour of piles subject to a sinusoidal input giving the following expression of the stationary bending moment:

$$M = 2.7 \cdot 10^{-7} E_p d^3 \left(\frac{a_{\max,S}}{g}\right)\left(\frac{L}{d}\right)^{1,3}\left(\frac{E_P}{E_1}\right)^{0,7}\left(\frac{V_{s2}}{V_{s1}}\right)^{0,3}\left(\frac{H_1}{L}\right)^{1,25} \tag{3}$$

where L and d are the pile length and diameter, respectively; V_{s2}/V_{s1} the ratio between the shear wave velocities of the two soil layers; H_1 the depth of the first layer; and $a_{max,s}$ the maximum acceleration at the soil surface (fig. 1).

Mylonakis [5] presented an evolution of Dobry and O'Rourke model [11] that consider the thickness of the two soil layers and the dynamic nature of the seismic input. Mylonakis proposed calculating the kinematic moment through bending deformation of the pile ε_P (bending strain) that is generated in the fibres of the external radius r; in particular:

$$M = E_P I_P \left(\frac{\varepsilon_P}{\gamma_1} \right) \gamma_1 \; \Phi \; r \tag{4}$$

where

$$\frac{\varepsilon_P}{\gamma_1} = 1.5 \left(\frac{k_1}{E_P} \right)^{1/4} F \tag{5}$$

$$F = \frac{\left(C^2 - C + 1\right)\left[\left(2\lambda_1 h_1 - 1\right)C\left(C - 1\right) - 1\right]}{2C^4 \lambda_1 h_1} \tag{6}$$

$$\lambda_1 = \left(\frac{k_1}{4E_P I_P} \right)^{1/4}; \qquad k_1 = \frac{3G_1}{d} \tag{7}$$

being: Φ a factor that takes into account the seismic input frequency and varies between 1 and 1.2; and C the coefficient of Dobry and O'Rourke [11] (see expression (2)).

More recently Nikolaou et al. [13] proposed a new relationship that comes from numerous experimental tests. This enables the determination of maximum bending moment of the pile at the interface between two layers of different stiffnesses, in ideal conditions of stationary motion, with frequency next to the fundamental frequency of the deposit. In this case:

$$M_{\max} = 0.042 \tau_c d^3 \left(\frac{L}{d} \right)^{0,30} \left(\frac{E_P}{E_1} \right)^{0,65} \left(\frac{V_{s2}}{V_{s1}} \right)^{0,5} \tag{8}$$

where:

$$\tau_c = a_{\max,S} \rho_1 H_1 \tag{9}$$

and ρ_1 is the density of the superficial layer.

3 Soil-pile kinematic interaction modelling: strategy of analysis

This paper deals with the problem of a single pile embedded in a soil constituted of two layers of different stiffnesses, resting on an infinitely rigid base (fig. 1). The two layers of soil are considered as linear, elastic, viscous and isotropic materials. Specifically two cases (cases A2 and A3) treated by Maiorano and Aversa [4] are considered in the present paper. Table 1 shows the main features of the soil involved. The pile has a diameter d = 0.6 m and a length L = 12 m. It is also hypothesized that the head of the pile coincides with soil surface and that the pile has a linear, elastic isotropic behaviour characterized by E_p = 30000 MPa and a Poisson ratio ν = 0.2. Finally, the free and fixed head pile conditions are considered for both cases A2 and A3.

The seismic excitation is constituted by Tolmezzo accelerogram (1976) reduced to 1m/s^2 (fig.2). Cases A2 and A3 were analyzed by Maiorano and

Table 1: Characteristics of soil foundation.

	H_1 (m)	H_2 (m)	E_1 (MPa)	E_2 (MPa)	V_{S2}/V_{S1} -	ν -	β	ρ_1 kNs/m^4	ρ_2 kNs/m^4	ρ_1/ρ_2 -
A2	8	4	30	150	2	0.4	10%	1.63	2.04	0.79
A3	8	4	30	686	4	0.4	10%	1.63	2.33	0.69

The subscript "1" refers to the first layer of soil moving from soil surface and the subscript "2" to the second layer. Moreover, the symbols listed in the table have the following meanings for the single layer: H=thickness; E=Young modulus; V_S=shear wave velocity; υ=Poisson ratio; β=damping ratio; ρ=density.

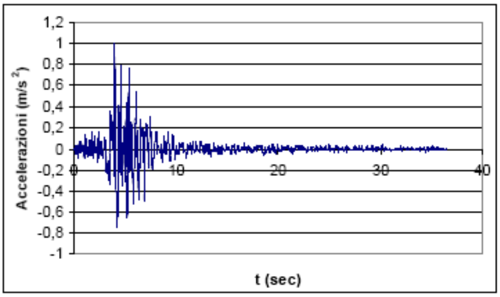


Figure 2: Accelerogram of Tolmezzo, 1976.

Aversa [4] using the finite element code VERSAT-P3D (Wu and Finn [21]), which is based on a simplified formulation of three-dimensional propagation wave equations for linear-elastic, viscous soil.

While in this paper the finite element ADINA (Bathe [22]) code is used. It is based on transient dynamic analysis. Two different 3D models are developed: in the former (fig. 3a) the soil is meshed with 8-node 3D-solid elements while the pile is meshed with beam elements; in the latter (fig. 3b) both the soil and the pile are meshed with 8-node 3D-solid elements.

In both cases, vertical boundaries are 12 m far from the pile, which is located at the centre of the mesh; the horizontal bottom boundary is 12 m far from the tip of the pile. Furthermore, on the horizontal bottom boundary vertical displacements are not allowable; similarly, on the two vertical boundaries along the "y" direction horizontal displacements in the "x" direction are not allowed.

Finally, on the two vertical boundaries along the "x" direction special constrain equations along the "y" direction are imposed. Specifically, each node of one of these boundaries must have the same displacement in "y" direction of its corresponding node in the opposite boundary. Two nodes are considered corresponding nodes if they are part of two opposite vertical boundaries and have the same distance from the horizontal bottom boundary and the same distance from the other two vertical boundaries.

Whole model is initially subjected to the "mass proportional" command to take into account the unit weight of the involved materials. Then the horizontal bottom boundary is subjected to a horizontal displacement time-history along the

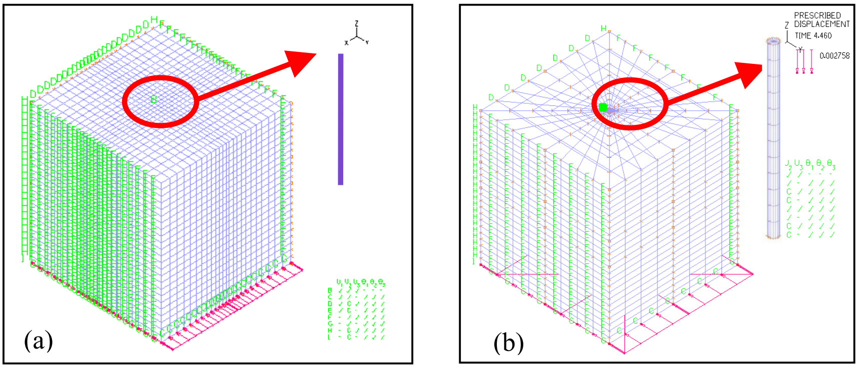


Figure 3: Adopted FEM models: (a) pile subdivided in beam elements; (b) pile subdivided in 3D-solid elements.

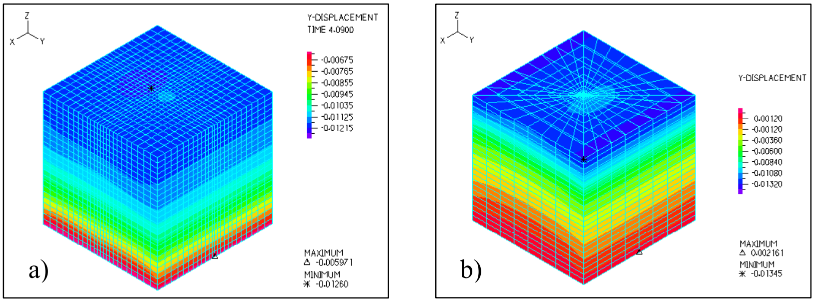


Figure 4: Horizontal displacement along the input motion direction: (a) pile meshed with beam elements (b) pile meshed with 3D-solid elements.

"y" direction (fig. 3). This displacement time-history is obtained from the accelerogram of fig.1, imposing that initial displacement and initial velocity are equal to zero.

3 Preliminary results

Fig. 4 shows at "y" horizontal displacements occurring at a generic time in the case of pile subdivided in beam elements (fig. 4(a)) and in the case of pile subdivided in 3D-solid elements (fig. 4(b)).

Fig. 5 shows stress distribution in the pile when it is modelled with 3D-solid elements. Only with this mesh stress distribution in any pile section can be accurately evaluated. Thus, considering the Navier beam theory it is possible to obtain kinematic moments along the pile.

Figs. 6(a) and 6(b) show kinematic moment distributions along the pile, considering the fixed head pile condition and the beam element meshing of pile. More precisely, figs. 6(a) and 6(b) refer to case A2 and case A3 of table 1, respectively.

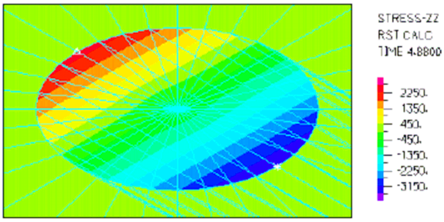
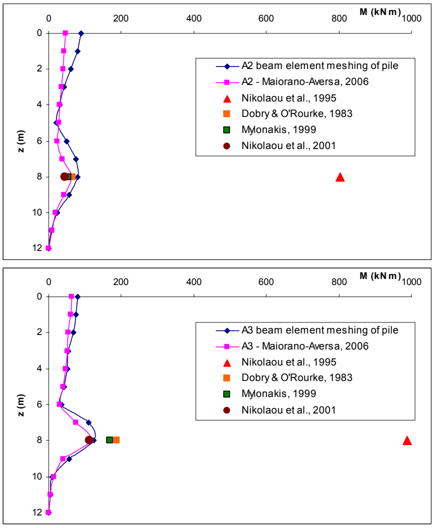


Figure 5: Stress distribution in a generic section of pile modelled with 3D-solid elements.



(a)

(b)

Figure 6: Kinematic moment distribution for fixed-head pile: (a) case A2; (b) case A3. Pile modelled with beam elements.

Figs. 6(a) and 6(b) also report the comparison with the results obtained from Maiorano and Aversa (4). In addition, as concerns the interface between the two soil layers, the comparison with the results achieved considering the Winkler models discussed in paragraph 2 are given. There is a very good agreement between the results obtained with the present FEM modelling and those obtained by Maiorano and Aversa [4]; as well as, at the soil layer interface, between the results obtained with the present FEM modelling and those obtained with Dobry and O'Rourke (11), Mylonakis (5) and Nikolaou et al. (13) models. Nikolaou et al. [12] model leads to an excessive overestimation of kinematic moment at the soil layer interface.

Moreover, figs. 7(a) and 7(b) report the comparison between the kinematic moments obtained by dividing the pile into beam elements and kinematic moments obtained by dividing the pile with 3D-solid elements. The two related distributions agree each other, but there are values higher if the pile is meshed with 3D-solid elements. Particularly, at the soil layer interface the moment obtained meshing the pile with 3-D solid elements has a higher value by 30% and 15% respectively for cases A2 and A3 compared to the moment obtained meshing the pile with beam elements.

Similar results on kinematic moments are obtained for free-head pile for cases A2 (Fig. 8(a)) and A3 (Fig. 8(b)).

(a)

(b)

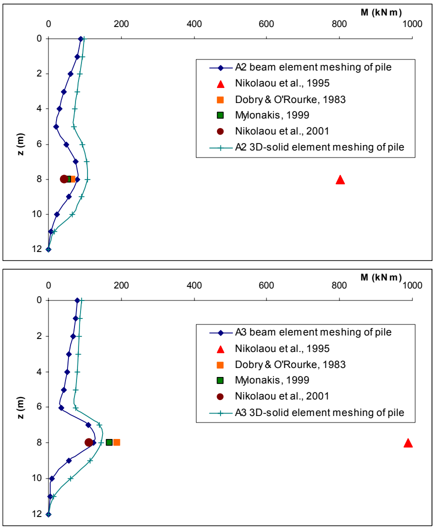
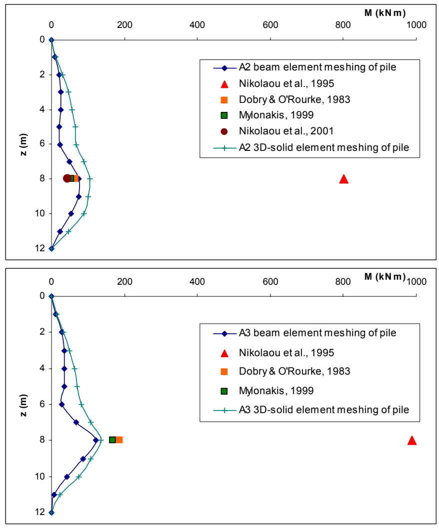


Figure 7: Kinematic moment distribution for case A2 (a) and A3 (b): comparison between two different kinds of pile meshing.

4 Conclusions

Bending moments on foundation pile due to soil-pile kinematic interaction can cause severe damage on pile; because of that, in the present paper these moments on a single pile are evaluated by means of a 3-D FEM approach. The pile is supposed to be embedded into two soil layers, characterised by different stiffnesses. Two kinds of pile meshing are considered: i) in the first one the pile is meshed using beam elements; in the second one the pile is meshed using 3-D solid elements. In both the cases the results are compared with those obtained with other simpler FEM analyses, as well as with those obtained with simplified BDWF procedures, which only give the kinematic bending moment at the interface between the two soil layers. Both the proposed FEM models (pile meshed with beam elements and pile meshed with 3D-solid elements) offer results in good agreement with those obtained recently by the other researchers. Nevertheless, the proposed FEM model based on a pile meshing with 3-D solid elements takes in account the actual geometry of the pile and provides bending moments slightly higher than those estimated with the simpler meshing of the pile with beam elements.

Finally, the present FEM approach can be easily extended to perform the analysis of both kinematic and inertial interaction phenomena, avoiding the simplified “method of substructures”.



(a)

(b)

Figure 8: Kinematic moment distribution for free-head pile: (a) case A2; (b) case A3.

Reference

[1] EC8-Part 5. Design of structures for earthquake resistance. Part 5: Foundations, retaining structures and geotechnical aspects. Technical Committee CEN/TC250, Ref. No. prEN 1998-5: Final draft. 2003.

[2] D.M. 14/01/2008 – Nuove norme tecniche per le costruzioni. 2008; published in Gazzetta Ufficiale n.29 del 4 February 2008.

[3] AGI – Aspetti geotecnici della progettazione in zona sismica, Linee Guida – Edizione provvisoria, March 2005.

[4] Maiorano, R.M.S., Aversa S. Importanza relativa di interazione cinematica ed inerziale nell’analisi dei pali di fondazione sotto azioni sismiche- Atti del V Convegno Nazionale Ricercatori ing. geotecnica, Bari, 15-16 sett. 2006.

[5] Mylonakis, G. Seismic Pile Bending at Deep Interfaces. Report GEL-99-01, Geotechnical Laboratory, City College of New York. 1999.

[6] Cairo, R., Dente, G. Kinematic interaction analysis of piles in layered soils. XIV European Conference on Soil Mechanics and Geotechnical Engineering - Madrid, 25th September 2007

[7] Gazetas G., Mylonakis G. Seismic soil-structure interaction: new evidence and emerging issue. Proc. 3rd Conf Geotechnical Earthquake Engineering and Soil Dynamics, ASCE, Seattle, pp.1119-1174. 1998.

[8] Margason, E. Pile bending during earthquakes, Lecture, March 6, 1975, ASCE/UC-Berkeley seminar on design construction & performance of deep foundation. 1975.
[9] Margason, E., Halloway, D. M. Pile design during earthquakes. Proc. 6th Wld Conf. Earthq. Engng, New Delhi, 237-243. 1977.
[10] NEHRP. Recommended provisions for seismic regulations for new buildings and other structures. Building Seismic Safety Council, Washington.1997.
[11] Dobry R., O'Rourke M.J. Discussion on 'Seismic response of end-bearing piles' by Flores-Berrones R., Whitman R.V. J. Geotech. Engng Div., ASCE, 109, pp. 778-781. 1983.
[12] Nikolaou A. S., Mylonakis G., Gazetas G. Kinematic bending moments in seismically stressed piles. Report NCEER-95-0022, National Center for Earthquake Engineering Research. Buffalo: New York. 1995.
[13] Nikolaou A. S., Mylonakis G., Gazetas G., Tazoh T. Kinematic pile bending during earthquakes analysis and field measurements. Géotecnique 51, n° 5, pp. 425-440. 2001.
[14] Sica, S., Mylonakis, G., Simonelli, A. L. Kinematic bending of piles: Analysis vs. code provisions. 4th International Conference on Earthquake Geotechnical Engineering, June 25-28, Paper No. 1674. 2007.
[15] Castelli, F., Maugeri, M., Mylonakis, G. Numerical analysis of kinematic soil-pile interaction. "2008 Seismic Engineering International Conference MERCEA'08", Reggio Calabria and Messina (Italy), 8-11 July 2008.
[16] Conte, E., Dente, G. Effetti dissipativi nella risposta sismica del palo singolo. Conv. Sul tema: Deformazioni del terreni ed interazione terreno-struttura in condizioni di esercizio, Mondelice, pp. 19-38. 1988.
[17] Conte, E., Dente, G. Il Comportamento sismico del palo di fondazione in terreni eterogenei. XVII Convegno Nazionale di Geotecnica – 1989.
[18] Castelli, F., Maugeri, M. Numerical analysis for the dynamic response of a single pile. Proc. XIV European Conference on Soil Mechanics and Geotechnical Engineering (ECSMGE), Madrid, 24-27 September 2007.
[19] Mairoano R.M.S., Aversa S. and Wu G. Effects of soil non-linearity on bending moments in piles due to seismic kinematic interaction. 4th Int. Conf. on Earthquake Geotech. Eng., Paper No. 1574. 2007
[20] Cairo R., Conte E., Dente G. R. *Nonlinear seismic response of single piles*. Atti del convegno "2008 Seismic Engineering International Conference MERCEA'08", Reggio Calabria and Messina (Italy), 8-11 July 2008.
[21] Wu, G., Finn, W. Dynamic Elastic Analysis of pile foundations using finite element method in the frequency domain. Can. Geotech. J., 34(1), 34-43. 1997.
[22] Bathe, K.J. Finite Element Procedures. Prentice Hall. 1996.