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SOIL-PILE KINEMATIC INTERACTION: NEW PERSPECTIVES FOR EC8 IMPROVEMENT

Roberto Cairo [a], Enrico Conte [a], Giovanni Dente [a],
Stefania Sica [b], Armando Lucio Simonelli [b]

[a] *University of Calabria, Rende (CS), Italy, cairo@dds.unical.it*
[b] *University of Sannio, Benevento, Italy, alsimone@unisannio.it*

ABSTRACT
Kinematic interaction between soil and structure originates from the incompatibility of the seismic free-field motion and the displacements of a more rigid embedded foundation. Foundation piles will actually experience deformations not only due to the loads directly transmitted by the superstructure, but also due to the passage of the seismic waves through the surrounding soil. In particular, bending moments may reveal quite important and, under certain circumstances, should be taken into account in the design, as prescribed by recent seismic codes (EN 1998-5; D.M. 14.1.2008). In this paper, the main results of a theoretical study are illustrated and some preliminary elements to be considered for seismic building codes are suggested.

KEYWORDS
Piles, kinematic interaction, code, simplified approach, bending moment.

1 INTRODUCTION

During strong earthquakes foundation piles tend to significantly modify soil deformations, since they oppose to the seismic motion of the ground. The interplay between soil and structure makes the motion at the base of the superstructure to deviate from the free-field motion, and the piles to be subjected to additional bending, axial and shearing stresses. The bending moments, usually referred to as “kinematic” ones, may result somewhat important even in the absence of the superstructure.
The kinematic interaction between soil and piles has been studied by many researchers (Fan et al., 1991; Gazetas et al., 1992; Kaynia and Mahzooni, 1996; Poulos and Tabesh, 1996; Mylonakis, 2001; Nikolaou et al., 2001; Saitoh, 2005; Cairo and Dente, 2007; Sica et al., 2007; Simonelli and Sica, 2008; Maiorano et al., 2009, and others). In spite of the big effort on such a topic, kinematic interaction is rarely accounted for in practical design. Modern seismic codes have, however, acknowledged the importance of kinematic interaction and demand piles to be designed also accounting for soil deformations arising from the passage of seismic waves. Two main issues should be also addressed by a building code:

- when has kinematic interaction to be considered (or, conversely, when can it be neglected)?;
- how has kinematic interaction to be analysed?

At present, the evidence collected is far from providing a definitive answer to the former question, but it is adequate to indicate what has still to be done for this issue. Eurocode 8 (EN 1998-5) suggests that kinematic effects should be taken into account when all the following conditions simultaneously exist: 1) seismicity of the area is moderate or high (specifying that moderate or high-seismicity areas are characterized by a peak ground acceleration a_gS>0.1g, where a_g is the design ground acceleration on type A subsoil and S is the soil factor); 2) subsoil type is D or worse, characterized by sharply different shear moduli between consecutive layers; 3) the importance of the superstructure is of III or IV class (e.g. schools, hospitals, fire stations, power plants, etc). The recent Italian building code (D.M. 14.1.2008) provides quite similar indications concerning the kinematic bending moments in piles.
The topic of this paper is to illustrate the key aspects of pile kinematic interaction and to individuate some preliminary elements to be considered for technical codes.

2 METHODS OF ANALYSIS

2.1 Overview of existing methods

Available methods for the analysis of kinematic soil-pile interaction may be classified into three groups: numerical approaches (FEM, BEM), Winkler methods (BDWF), simplified formulations. The finite element method (Wu and Finn, 1997; Cai et al., 2000; Kimura and Zhang, 2000; Maheshwari et al., 2004) provides a powerful and versatile technique, since some important effects such as soil nonlinearity and heterogeneity may be directly accounted for. Nevertheless, this method is generally very expensive from a computational viewpoint, since it requires suitable boundaries conditions being introduced to simulate the radiation damping effect. In such a context, a more attractive approach is represented by the boundary element technique (Kaynia and Kausel, 1982; Mamoon and Banerjee, 1990; Cairo and Dente, 2007). It only needs the discretization of the interfaces and permits the condition of wave propagation towards infinity to be automatically satisfied. This technique is generally formulated in the frequency domain and, in principle, is valid only under the assumption of material linear behaviour.
The methods based on the Winkler foundation model (Novak, 1974; Flores-Berrones and Whitman, 1982; Kavvadas and Gazetas, 1993) prove quite accurate and computationally time saving. They allow nonlinear behaviour of the soil to be easily incorporated if solution is envisaged in the time domain (Boulanger et al., 1999; El Naggar et al., 2005; Maheshwari and Watanabe, 2006; Cairo et al., 2008). According to this method, the pile is modelled as a linearly elastic beam, with length L and diameter d, discretized into segments connected to the surrounding soil by springs and dashpots, which provide the interaction forces in the lateral direction (Figure 1).
As a first approximation, the spring stiffness k may be considered to be frequency-independent and expressed as a multiple of the local soil Young's modulus E_s (Kavvadas and Gazetas, 1993). The dashpot coefficient c represents both material and radiation damping. The latter one may be computed using the analogy with one-dimensional wave propagation in an elastic prismatic rod of semi-infinite extent (Gazetas and Dobry, 1984).
For harmonic vertically propagating S-waves, the governing differential equation of the pile response is:

$$E_p I_p \frac{\partial^4 u_p}{\partial z^4} + m_p \frac{\partial^2 u_p}{\partial t^2} = (k + i\omega c)(u_{ff} - u_p) \tag{1}$$

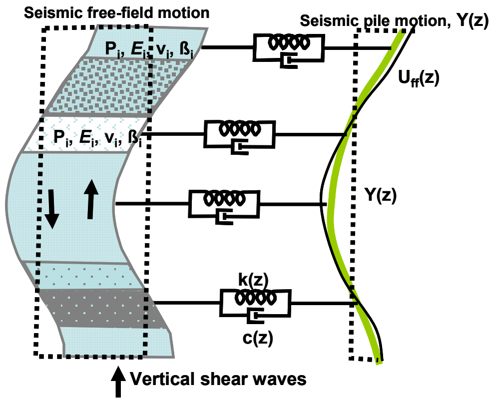

Figure 1. Beam on Dynamic Winkler Foundation model (BDWF).

where E_pI_p is the pile flexural rigidity, m_p the pile mass per unit length, u_p is the amplitude of the pile lateral displacement, u_{ff} the free-field soil displacement, ω the circular frequency of the motion, z is the depth, t the time and $i=(-1)^{0.5}$. Pile response in the time domain is then attained through standard Fourier transformations. Some results obtained using the Winkler approach (BDWF), as proposed by Mylonakis et al. (1997), will be presented in a subsequent section of the paper.

2.2 Approximate methods

Closed-form expressions (Dobry and O'Rourke, 1983; Nikolaou and Gazetas, 1997; Mylonakis, 2001; Nikolaou et al., 2001) are available in literature for approximately computing the maximum steady-state bending moment at the interface between two layers. These approaches have been derived by modelling the pile as a beam on a Winkler foundation and are based on the following simplified assumptions: each soil layer is homogeneous, isotropic and linearly elastic; the soil is subjected to a uniform static shear stress field; the pile behaves as a linear-elastic semi-infinite beam; the embedded length of the pile in each layer is greater than the so-called "active length". This latter is usually expressed by the equation (Randolph, 1981):

$$L_a = 1.5d\left(\frac{E_p}{E_s}\right)^{0.25} \tag{2}$$

The simplified design procedure proposed by Dobry and O'Rourke (1983) permits to compute the bending moment at the interface between two layers as:

$$M = 1.86(E_p\, I_p)^{3/4}(G_1)^{1/4}\,\gamma_1\, F \tag{3}$$

where G_1 and γ_1 are the shear modulus and shear strain in the upper soil layer, respectively; F is a function of the ratio $c=(G_2/G_1)^{0.25}$, with G_2 being the shear modulus of the lower layer, and expressed by:

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

An improvement of the Dobry and O'Rourke (1983) solution has been achieved by Mylonakis (2001). Using more suitable dynamic parameters, the following expression has been derived:

$$M = 2\frac{E_p I_p}{d}\left(\frac{\varepsilon_p}{\gamma_1}\right)\gamma_1 \tag{5}$$

in which ε_p indicates the pile peak bending strain and γ_1 denotes the soil shear strain at the layer interface. The ratio of these parameters represents a sort of "strain transmissibility" function, which is strongly frequency-dependent. If this aspect is neglected, the strain transmissibility function takes into account only pile-soil interaction effects and is expressed as:

$$\left(\frac{\varepsilon_p}{\gamma_1}\right)_{\omega=0} = \frac{c^2 - c + 1}{2c^4}\left(\frac{H_1}{d}\right)^{-1}\left\{\left[4.7\left(\frac{E_1}{E_p}\right)^{9/32}\frac{H_1}{d} - 1\right]c(c-1) - 1\right\} \tag{6}$$

where H_1 and E_1 are the thickness and the Young's modulus of the upper layer, respectively. The effect of frequency may be introduced in terms of the ratio:

$$\Phi = \left(\frac{\varepsilon_p}{\gamma_1}\right)\Big/\left(\frac{\varepsilon_p}{\gamma_1}\right)_{\omega=0} \tag{7}$$

which is a function of H_1/d, G_1/G_2, E_p/E_1. Nevertheless, in the range of frequencies of interest Φ is generally less than 1.25.

A fitted formula has been proposed by Nikolaou et al. (2001) for harmonic excitation. It is based on the maximum shear stress τ_c induced at the layer interface by the free-field motion. The resulting expression is:

$$M = 0.042\,\tau_c\, d^3\left(\frac{L}{d}\right)^{0.30}\left(\frac{E_p}{E_1}\right)^{0.65}\left(\frac{V_{s2}}{V_{s1}}\right)^{0.50} \tag{8}$$

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

In order to account for the transient nature of the seismic excitation, the authors have introduced a reduction factor depending on the duration of the accelerograms in terms of the effective number N_c of cycles in the record, the relative frequency characteristics between earthquake and soil deposit, the effective damping of the soil-pile system. Two simplified expressions are given:

$$\eta = \begin{cases} 0.04N_c + 0.23 & \text{for } T_1 \approx T_p \\ 0.015N_c + 0.17 \approx 0.2 & \text{for } T_1 \neq T_p \end{cases} \tag{9}$$

where T_p represents the predominant period of the ground motion and T_1 the fundamental period of the deposit.

A crucial issue in the use of these approximate approaches is the evaluation of the uniform soil shear strain γ_1 in the upper layer and the maximum shear stress τ_c at the interface. As

suggested by Mylonakis (2001), if the seismic peak acceleration $a_{\max s}$ is specified at the soil surface, γ_1 can be computed using the expression suggested by Seed and Idriss (1982) for liquefaction problems:

$$\gamma_1 = (1 - 0.015H_1)\frac{\rho_1 H_1}{G_1} a_{\max s} \tag{10}$$

where, in addition to the soil parameters already defined, the mass density of the upper layer ρ_1 has been introduced. Equivalently, the maximum shear stress τ_c at the interface may be roughly estimated as (Nikolaou and Gazetas, 1997; Nikolaou et al., 2001):

$$\tau_c = a_{\max s}\rho_1 H_1 \tag{11}$$

In the next section, the accuracy of these simplified approaches will be investigated.

3 MAIN RESULTS

Several analyses have been performed referring to a fixed-head pile with length L=20 m, diameter d=0.6 m, Young's modulus E_p=2.5·10^7 kN/m², and mass density ρ_p=2.5 Mg/m³ (Figure 2). The pile is embedded in a two-layered subsoil which is 30 m thick and underlined by a stiffer bedrock. Soil shear stiffness contrast has been changed as a function of the shear wave velocities V_{s1} and V_{s2} of the two layers, in order to reproduce a subsoil of type D or C (D.M. 14.1.2008). Poisson's ratio and mass density of the soil are: ν_s=0.4, ρ_s=1.9 Mg/m³. The shear wave velocity of the rock is 1200 m/s. The analyses have been performed by adopting 18 Italian accelerograms (Table 1) scaled in amplitude to provide a rock peak acceleration consistent with the seismic zone considered. These records are provided by the database SISMA (Scasserra et al., 2008). In Table 1, the frequency content of the input motion is quantified through the predominant period T_p, corresponding to the maximum spectral acceleration in an acceleration response spectrum (computed for 5% viscous damping) and through the mean period, T_m, as defined by Rathje et al. (1998) on the basis of the Fourier spectrum of the signal. Actually, T_m should provide a better indication of the frequency content of the recordings because it averages the spectrum over the whole period range of amplification.

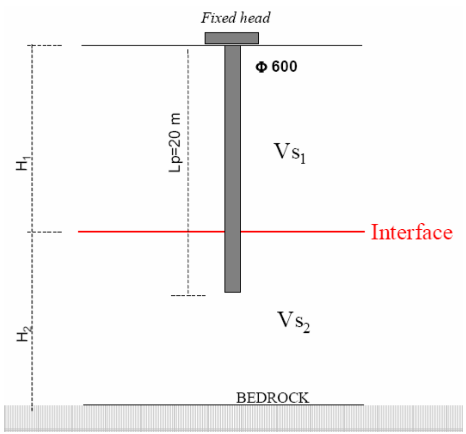

Figure 2. Reference scheme adopted in the analyses.

The main results of the whole parametric study, described elsewhere (Sica et al., 2007; Simonelli and Sica, 2008), are reported in the following. It is worth noting that the behaviour of the soil-pile system is assumed to be linearly elastic.

Table 1. Seismic records provided by the database SISMA and used in the analyses.

Label	Station name	Earthquake	Comp.	Date (d/m/y)	T_p (s)	T_m (s)
A-TMZ270	Tolmezzo-Diga Ambiesta	Friuli	EW	6.5.1976	0.64	0.500
A-TMZ000	Tolmezzo-Diga Ambiesta	Friuli	NS	6.5.1976	0.26	0.395
A-STU270	Sturno	Campano Lucano	EW	23.11.1980	0.20	0.845
A-STU000	Sturno	Campano Lucano	NS	23.11.1980	0.38	0.661
A-AAL018	Assisi-Stallone	Umbria Marche	NS	26.9.1997	0.32	0.333
E-NCB090	Nocera Umbra-Biscontini	Umbria Marche (aftershock)	EW	6.10.1997	0.12	0.172
E.NCB000	Nocera Umbra-Biscontini	Umbria Marche (aftershock)	NS	6.10.1997	0.14	0.165
R-NCB090	Nocera Umbra-Biscontini	Umbria Marche (aftershock)	EW	3.4.1998	0.18	0.180
J-BCT000	Borgo-Cerreto Torre	Umbria Marche (aftershock)	NS	14.10.1997	0.10	0.167
J-BCT090	Borgo-Cerreto Torre	Umbria Marche (aftershock)	EW	14.10.1997	0.16	0.208
E-AAL018	Assisi-Stallone	Umbria Marche (aftershock)	EW	6.10.1997	0.22	0.242
B-BCT000	Borgo-Cerreto Torre	Umbria Marche	NS	26.9.1997	0.08	0.154
B-BCT090	Borgo-Cerreto Torre	Umbria Marche	EW	26.9.1997	0.12	0.198
TRT000	Tarcento	Friuli (aftershock)	NS	11.9.1976	0.10	0.215
C-NCB000	Nocera Umbra-Biscontini	Umbria Marche (aftershock)	NS	3.10.1997	0.04	0.128
C-NCB090	Nocera Umbra-Biscontini	Umbria Marche (aftershock)	EW	3.10.1997	0.12	0.154
R-NC2090	Nocera Umbra 2	Umbria Marche (aftershock)	EW	3.4.1998	0.18	0.184
R-NC2000	Nocera Umbra 2	Umbria Marche (aftershock)	NS	3.4.1998	0.16	0.152

Figure 3 presents the envelopes of the maximum kinematic bending moments with depth, as computed in the time domain using the Winkler approach by Mylonakis et al. (1997), for each of the selected 18 accelerograms (scaled to the same peak ground acceleration of 0.35g). The thickness of the layers is assumed to be H_1=H_2=15 m; the shear wave velocities of the upper and the lower layers are V_{s1}=100 m/s and V_{s2}=400 m/s, respectively. On the basis of the average shear wave velocity $V_{s,30}$=160 m/s, the soil profile under consideration can be classified as type D. The damping ratio of the soil is β_s=0.10.

Three grey zones are also displayed corresponding to the range of reinforced concrete pile yielding moments for typical reinforcements of the cross section (8ϕ16, 24ϕ12 and 12ϕ30) and magnitude of the axial force in the pile. For each reinforcement the lower limit of the grey zone represents the cross section yielding moment corresponding to zero axial force while the higher one to an axial force equal to 1200 kN.

From the results of this study, the following remarks may be made:

- the maximum kinematic bending moment generally occurs at the soil layer interface;
- at the soil layer interface, the kinematic bending moment dramatically increases when the shear wave velocity contrast between the bottom and top layer Vs_2/Vs_1 increases from 2 to 4, for subsoil types D and C;
- for subsoil profiles corresponding to class D, the computed kinematic bending moments may be well above the assumed yielding moments of the pile cross section.

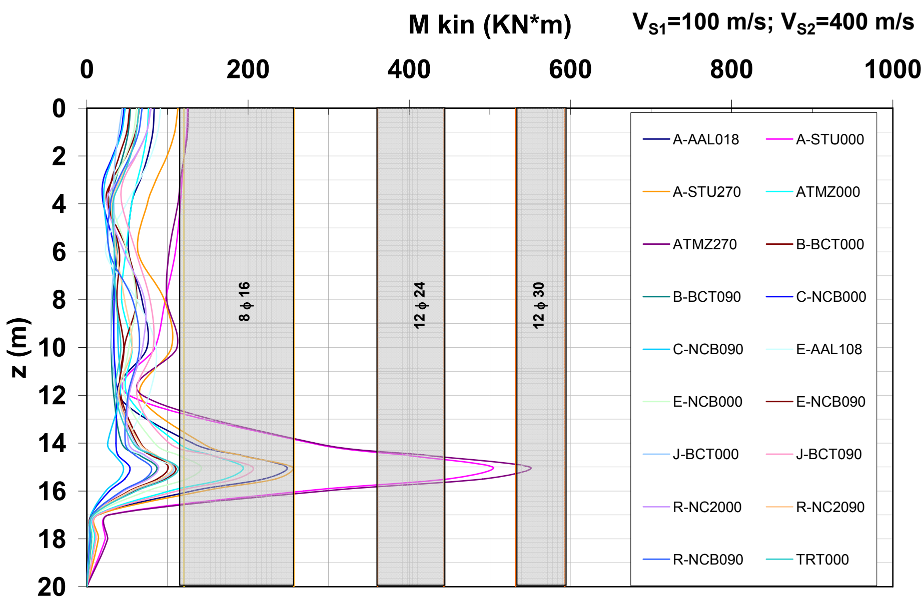

Figure 3. Kinematic bending moments for two-layer soil profile of type D and with H_1=H_2=15 m.

3.1 Effect of the frequency content of the earthquake

As a further interpretation of the results previously shown, the ratio between the period T_{input} of the input motion (in terms of T_p and T_m) and the fundamental period T_{soil} of the subsoil in hand (in the specific case, T_{soil}=0.62 s) is presented in Figure 4, for all the considered seismic records. It is worth noting that the accelerograms characterized by values of T_p/T_{soil} or T_m/T_{soil} close to unity provide the higher kinematic moments in Figure 3, due to the occurrence of a resonance phenomenon. This confirms what pointed out by Nikolaou et al. (2001) by frequency domain analyses: maximum effects of kinematic bending in piles occur at the fundamental period of the subsoil. Conversely, all accelerograms having T_p/T_{soil} or T_m/T_{soil} below 0.5, induce lower kinematic moments in the pile.

These results suggest that a “critical band” of the ratio $T_{\text{input}}/T_{\text{soil}}$ in which kinematic effects could be important, may be individuated (Figure 4). This observation could be a possible criterion to select significant records from a database of local seismic events. Therefore, if the acceleration time-histories are provided for a given seismic zone (in addition to the peak ground acceleration), the “susceptibility of the site with the associated waveforms” to induce significant kinematic bending in piles may be established on the basis of the following criterion:

- if the ratio $T_{\text{input}}/T_{\text{soil}}$ is external to the critical band, site susceptibility is low, and only inertial interaction should be accounted for;
- if the ratio $T_{\text{input}}/T_{\text{soil}}$ is internal to the critical band, site susceptibility is high, and kinematic interaction should be analysed in addition to inertial interaction.

In the latter case, the analysis tool should be consistent to the one adopted in the design of the overall structure.

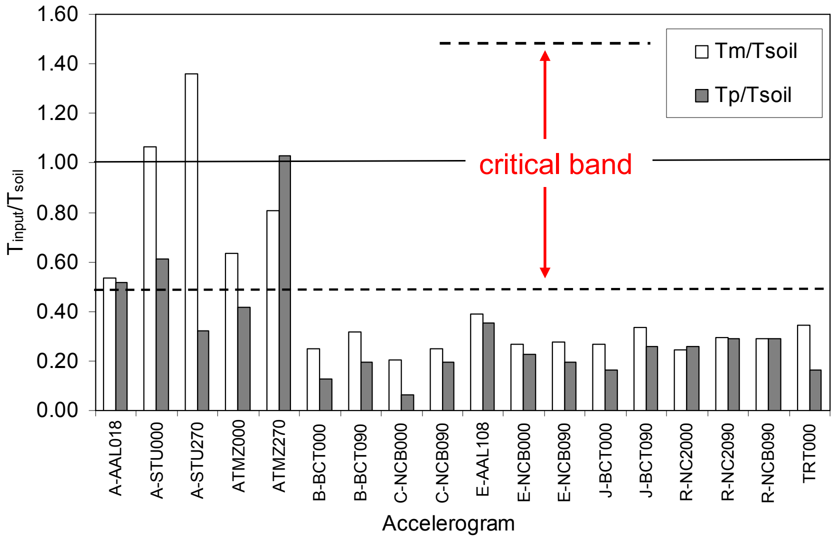

Figure 4. Ratio among the predominant period of the input motion and the fundamental period of the soil.

3.2 Kinematic moments computed with simplified methods

The simplified methods provided by Dobry and O'Rourke (1983), Mylonakis (2001), and Nikolaou et al. (2001) have been applied to compute the kinematic moments at the interface of the above specified two-layered subsoil, in order to compare their predictions to those obtained numerically by the BDWF approach of Mylonakis et al. (1997).

The formulas by Dobry and O'Rourke (1983) and Mylonakis (2001) have been applied by adopting both the shear strain at the bottom of the first layer γ_1 provided by Eq. (10) (Figure 5a) and the value of γ_1 directly computed with SHAKE (Schnabel et al., 1972) or EERA (Bardet et al., 2000) for each selected accelerogram (Figure 5b). At the same way, the equation derived by Nikolaou et al. (2001) has been applied by adopting both the shear stress at the interface using Eq. (11) as suggested by the authors (Figura 5a), and the value directly provided by EERA (Figura 5b). The formula of Nikolaou et al. (2001) has been applied without introducing any corrective factor η.

From Figure 5a and 5b, it emerges that if literature closed-form solutions - Dobry and O'Rourke (1983), Mylonakis (2001), and Nikolaou et al. (2001) - are adopted with the values of γ_1 and τ_c provided by the simplified approaches, all formulas overestimate the kinematic moments with respect to the values computed numerically by BDWF analyses. Conversely, if γ_1 or τ_c are derived from a free-field analysis, carried out with SHAKE or EERA, the kinematic moments provided by the literature formulas are quite close to those computed numerically (Figure 5b). Similar results have been obtained also by Maiorano et al. (2009).

In short, it seems well-established that it is more suitable to apply the literature formulas for estimating pile kinematic moment at the interface in combination with free-field analyses (with SHAKE or EERA) in order to get the proper value of γ_1 or τ_c, especially when the interface is quite deep. As well-known, in such case the formula provided by Seed & Idriss (1982) for computing γ_1 (or, equivalently, τ_c) is no longer reliable.

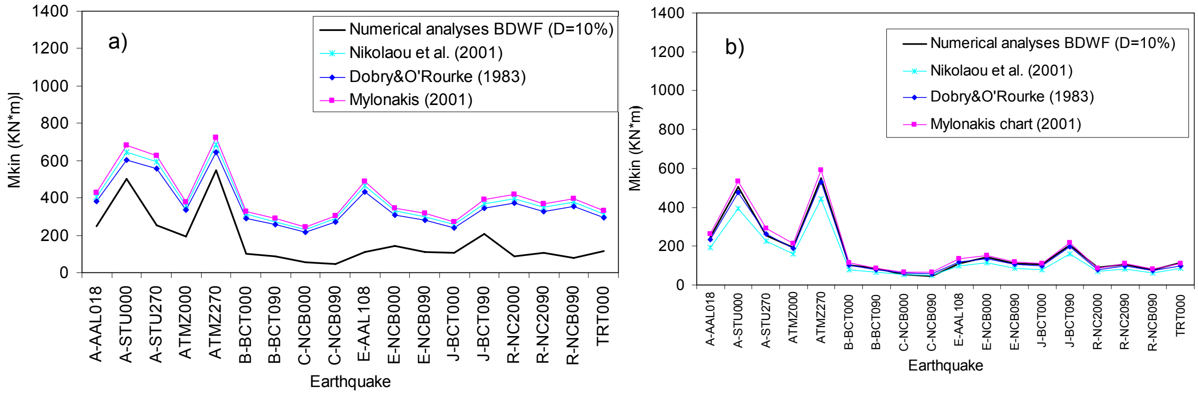

Figure 5. Kinematic moment at the interface computed with approximate formulas and the BDWF model.

3.3 *An alternative simplified approach*

Recently, Cairo et al. (2009) have suggested an alternative procedure that may be promptly used in current practice. This procedure (that is fully analytical) is very simple to use and needs only two parameters for defining the seismic motion: the peak ground acceleration and the mean period of the excitation. The former is directly provided by the code; the latter may be assumed on the basis of some suitable regression equations available in the literature (Rathje et al., 2004; Ausilio et al., 2007). In the proposed procedure, Eq. (8) by Nikolaou et al. (2001) has been considered the most suitable, and a "corrective" factor δ has been introduced. This latter is defined as the ratio of the maximum pile bending moment in the time domain, calculated using the BEM approach developed by Cairo and Dente (2007), to the kinematic moment obtained by Eq. (8), in which the peak ground acceleration is computed by site response analyses. The results of Figure 6 show the factor δ, calculated with reference to the case-studies previously documented, versus the ratio T_1/T_m of the fundamental period of the soil deposit to the mean period of the seismic excitation. A linear relationship between δ and T_1/T_m can be obtained with reference to the 95th percentile of the distribution. This line is described by the following expression:

$$\delta = 1.31 - 0.20\frac{T_1}{T_m} \tag{12}$$

This relation may be considered to evaluate the expected maximum bending moment. As a first approximation (Mylonakis, 2001) the fundamental period of the deposit may be evaluated by the equation:

$$T_1 = \frac{4H_1}{V_{s1}} \tag{13}$$

where H_1 and V_{s1} are the first layer thickness and shear wave velocity, respectively. Therefore, the maximum pile bending moment M_{kin} at the interface between two soil layers may be computed by:

$$M_{kin} = \delta \cdot M \tag{14}$$

with δ and M provided by Eq. (12) and Eq. (8), respectively.

As an example, this procedure has been applied to a case-study proposed by Nikolaou and Gazetas (1997) and recalled by Cairo and Dente (2007). A concrete pile is embedded 9.5 m into a top layer of soft clay and 6 m into a deep layer of dense sand. The shear wave velocities for the upper and lower layers are 80 and 330 m/s, respectively. The soil deposit is 30 m thick and rests on a rigid bedrock. The pile has E_p=25 GPa, d=1.3m, L=15.5 m, ρ_p=2.5 Mg/m^3. Two actual accelerograms, scaled to 0.10g peak acceleration, have been used as excitation at the rock level. In Figure 7 the results obtained by Eq. (14) are compared with the envelopes of the peak moments computed by Nikolaou and Gazetas (1997) and by Cairo and Dente (2007) using different methods. As can be seen, the agreement between the results is quite satisfactory.

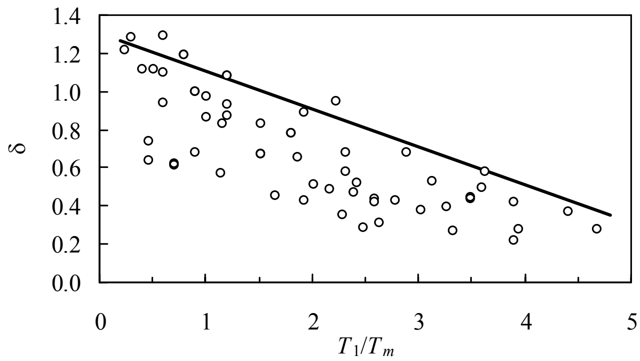

Figure 6. Corrective factor δ as a function of the ratio of the fundamental period T_1 of the soil deposit to the mean period T_m of the earthquakes.

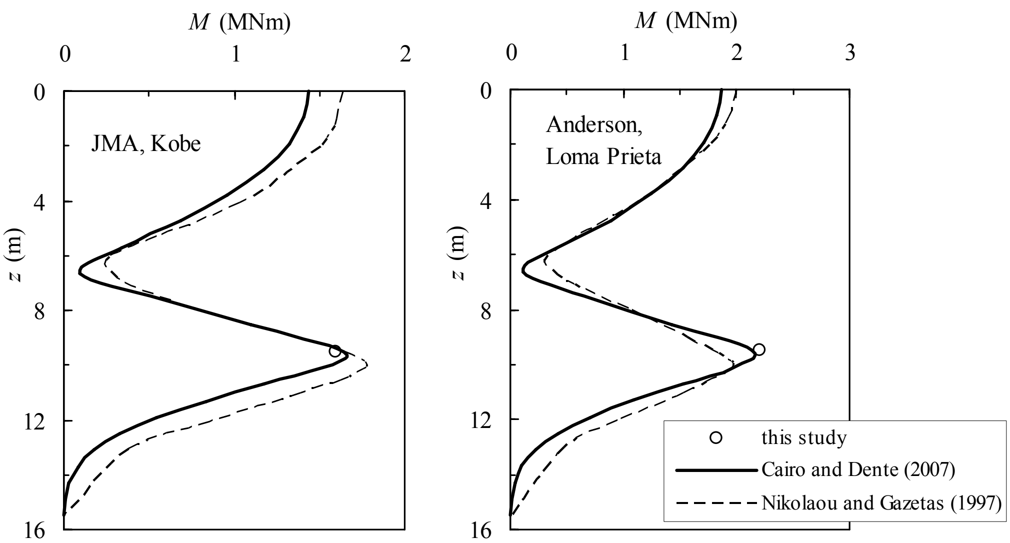

Figure 7. Envelopes of time-domain moments in a pile and peak kinematic moments at the interface.

4 CONCLUSIONS

In the paper some of the results from a comprehensive parametric study performed on a single fixed-head pile, embedded in two-layered soil deposits, have been illustrated. The analyses have been carried out using the Winkler-type model provided by Mylonakis et al. (1997), the

BEM formulation developed by Cairo and Dente (2007), and some simplified closed-form expressions available in the specific literature for evaluating, in an approximate manner, the maximum kinematic moment in the pile at the interface between two soil layers. Italian seismic records have been used as input motion for the numerical analyses.
From the obtained results, the following conclusions may be drawn:
- in high seismicity zones and especially for subsoil type D, kinematic bending moments may be very high depending on the stiffness contrast between consecutive layers;
- the maximum effects of kinematic pile bending generally occur for input motions having the fundamental period close to that of the subsoil;
- the simplified methods here examined (Dobry and O'Rourke, 1983; Mylonakis, 2001; Nikolaou et al., 2001) tend to predict conservative moments at the subsoil interface, especially when the interface is deep. In such case it is better their use in combination with free-field response analyses to get predictions closer to the ones obtained from more rigorous approaches.

On the basis of these observations, a simple criterion has been suggested to roughly estimate site susceptibility to induce significant kinematic bending in piles, and a simplified procedure for crudely evaluating kinematic moment has been presented. Yet, caution is required before proposing suggestions to be incorporated into seismic building codes. The theoretical evidence derived from the analysis needs to be extended to a wider selection of schemes, and to be compared with further experimental evidence.

5 ACKNOWLEDGEMENTS

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