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KINEMATIC BENDING MOMENTS AT PILE HEAD IN LAYERED SOIL

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ABSTRACT

During seismic shaking soil movements force piles to deform, resulting in a complex interplay between the two systems commonly referred to as "kinematic interaction". This interaction generates internal forces along piles even in the absence of a superstructure. Current design practice usually takes into account only forces transmitted to the pile from the superstructure, thus neglecting kinematic interaction. Despite intense research in the topic, few contributions focus on kinematic effects at the pile head in the presence of a stiff pile cap restraining its rotation. The subject may be of importance,for the pile head is subjected both to relevant kinematic and inertial moments, the latter attenuating rapidly with depth.
The paper presents the results of an extensive parametric study leading to a simplified formula for evaluating kinematic moments at the pile head in a two-layer soil profile.

Keywords: kinematic interaction, bending, pile head, finite elements

INTRODUCTION

Kinematic interaction in pile foundations has been the subject of intense research (Flores-Berrones and Whitman, 1982; Dobry and O'Rourke, 1983; Kavvadas and Gazetas, 1993; Nikolaou et al., 2001; Mylonakis, 2001; Maiorano et al., 2009); post-earthquake investigations have confirmed the severity of the phenomenon (Mizuno, 1987). The accumulated evidence has been recognized by recent seismic Codes such as Eurocode 8 (EN 1998-5, 2003) and the Italian Code (NTC DM 14/01/2008). These codes encourage designers to take into account kinematic effects - yet only under specific conditions. For example Eurocode 8 states that: "*piles shall be designed to resist the following two types of action effects: (a) inertia forces from the superstructure... ; (b) kinematic forces arising from the deformation of the surrounding soil due to the passage of seismic waves*", and that: "*bending moments developing due to kinematic interaction shall be computed only when all of the following conditions occur simultaneously: (1) the ground profile is of type D, S1 or S2, and contains consecutive layers of sharply differing stiffness; (2) the zone is of moderate or high seismicity, i.e. the product a_gS exceeds 0.10g; (3) the supported structure is of class III or IV*". The Italian Code provides similar indications. Di Laora (2009) showed that these provisions should be improved, as kinematic effects may be important over inertial ones depending on the circumstances. Furthermore, Eurocode 8 clearly encourages performing seismic design of piles with reference to 'kinematic' and 'inertial' forces, thereby dividing the soil-foundation-structure interaction in two complementary phenomena in the realm of the substructure method (Gazetas, 1984;

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Nikolaou et al., 1995; Mylonakis et al., 1997). According to this method, the system may be analyzed by performing the following consecutive steps:

(1) a kinematic interaction analysis based on the assumption that the superstructure has zero mass. This step is aimed at providing: (a) a component of internal forces acting along piles and (b) the Foundation Input Motion (FIM), that may be different from the free-field soil motion;

(2) an inertial interaction analysis in which the structure, shaken by FIM, induce forces and displacements at the foundation, which, in turn, may be replaced by a set of springs and dashpots. Gazetas (1991) and Mylonakis et al. (2006) provided tables and charts for determining springs and dashpots for various subsoil conditions;

(3) a structural analysis of internal forces induced along piles by inertial interaction.

It is worthy of note that: some authors (Gazetas, 1984; Fan et al., 1991; Makris et al., 1996) argue that if seismic motion has low-frequency content, FIM is quite similar to the free-field motion. Di Laora (2009) provides some practical rules to identify the cases where the latter should be used instead of FIM in inertial interaction analyses. Hence, kinematic effects may be taken into account with reference to simplified procedures available in the literature.

KINEMATIC BENDING AT PILE HEAD IN HOMOGENEOUS SOIL

Kinematic bending moments along a pile may be interpreted as a superposition ot two counteracting phenomena: (1) the deflected shape soil tries to impose on the pile; (2) the resistance of the latter to the induced displacements due to its stiffness and inertia As ground response is more straightforward to determine (e.g., using an established computer program), the keypoint lies in the evaluation of pile-soil interaction. Using a Beam-on-Dynamic-Winkler-Foundation (BDWF) model, it is straightforward to show that, for a long fixed-head pile embedded in a homogeneous halfspace, the ratio of pile and soil curvature at the pile head is:

$$\frac{(1/R)_p}{(1/R)_s} = \Gamma \tag{1}$$

in which Γ is a dimensionless kinematic interaction parameter given by (Flores-Berrones and Whitman, 1982; Nikolaou et al, 2001):

$$\Gamma \cong \left[1 + \frac{4E_p I_p \left(\frac{\omega}{V_s}\right)^4}{k} \right]^{-1} \tag{2}$$

where E_p and I_p are the pile Young modulus and cross-sectional moment of inertia, V_s the propagation velocity of shear waves in the soil, ω the cyclic excitation frequency and k the modulus of the Winkler springs (modulus of subgrade reaction), which is approximately equal to the Young's modulus of the soil material. For fixed-head piles, the amplitude of the above curvature ratio, shown in Fig. 1, is not larger than 1 and decreases monotonically with increasing frequency because of the resistance of the pile to short wavelengths. Di Laora (2009) showed that for fixed-head piles, even for large pile-soil stiffness ratios (E_p/E_s up to 10000 or so), Γ starts to decrease at frequencies higher than those relevant to real earthquakes and, hence, curvature ratio at the time domain may be taken equal to 1 at least as a first approximation.

Mylonakis (1999) derived a closed-form expression for the curvature ratio for a fixed-head pile in a homogeneous layer over rigid rock. This solution reads:

$$\frac{(1/R)_p}{(1/R)_s} = \Gamma \left[1 + \left(\frac{q}{\lambda}\right)^2 \frac{\cos qh \left(\sinh \lambda h \cos \lambda h - \cosh \lambda h \sin \lambda h\right) + (q/\lambda)\cosh \lambda h \cos \lambda h \sin \lambda h}{\sinh 2\lambda h + \sin 2\lambda h} \right] \tag{3}$$
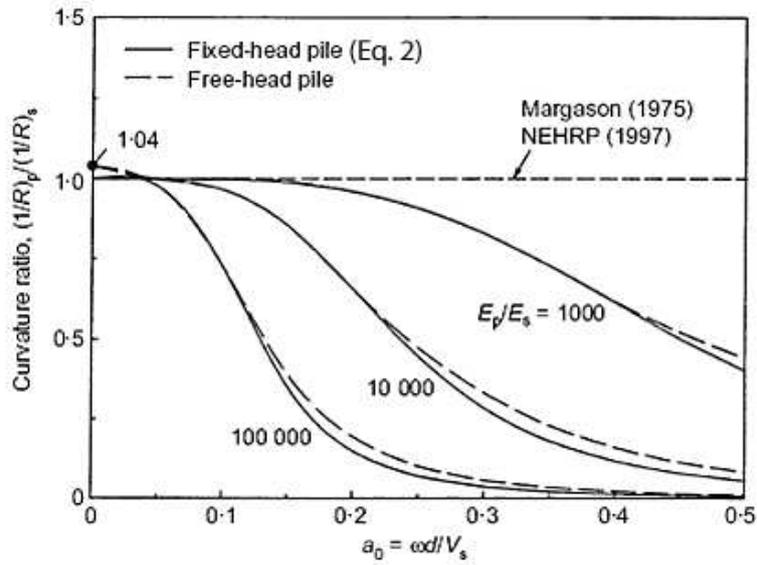

Figure 1. Curvature ratio for a pile embedded in a homogeneous halfspace (modified from Nikolaou et al., 2001).
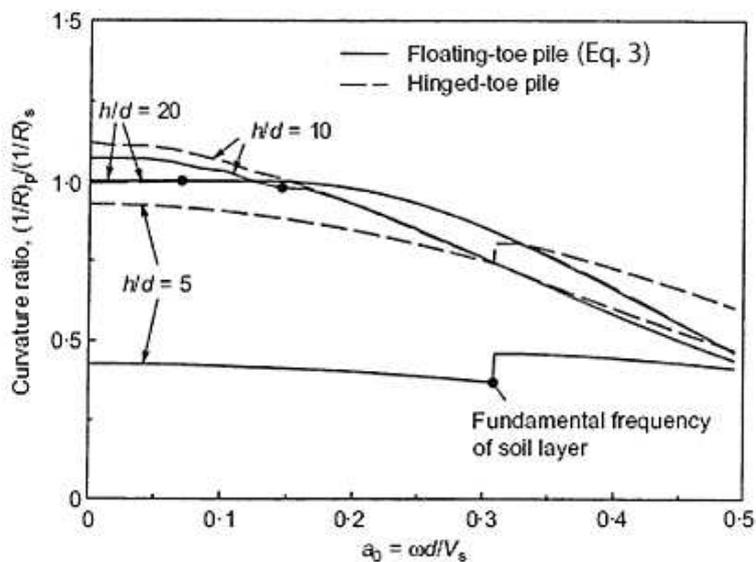

Figure 2. Curvature ratio for a pile embedded in a homogeneous layer over a rigid bedrock (modified from Nikolaou et al., 2001).

where

$$q = \omega/V_s \tag{4a}$$

and

$$\lambda = \left(\frac{k - m\omega^2}{4E_p I_p} \right)^{\frac{1}{4}} \tag{4b}$$

are the wavenumbers in the soil and the pile, respectively, with m being the pile mass per unit length. A graphical representation of Eq. 3 is provided in Fig. 2.

NUMERICAL ANALYSES FOR TWO-LAYER SOIL PROFILES

To explore pile-soil interaction at the pile head in presence of an interface separating two layers of different mechanical characteristics, numerical analyses were carried out by the authors using the commercial Finite Element (FE) code ANSYS. The analyses were performed in the frequency domain, based on a set of vibrational modes of equal damping having frequencies up to 12-15 times that of the fundamental mode and, in any case, less than 25 Hz. An FFT algorithm was employed to switch from time to frequency domain and vice versa. Linear elastic behaviour for both soil and pile was assumed.
The dimensions of the model have been selected to ensure that free-field response at the boundaries is not affected by waves spreading away from the pile-soil interface. Hence, vertical displacements are neglected along the lateral boundaries to simulate 1-dimensional conditions. Nodes at the base of the model are fully restrained to represent a rigid bedrock. In addition, proper restraints are imposed on vertical planes of symmetry and antisymmetry.
The geometrical scheme considered is depicted in Fig. 3a; the associated FE model is shown in Fig. 3b.

STATIC RESPONSE

In principle, bending at the head of a pile embedded in a two-layer deposit is different from the corresponding homogeneous case, as it is affected by the presence of the second layer. The difference is more pronounced for small interface depths, large pile diameters, large pile-soil stiffness contrasts and large stiffness contrasts between the soil layers. Figure 4 shows the distribution of curvature along a pile excited "statically" (i.e., at zero frequency) for different interface depths and layer stiffness contrasts.
It can be observed that curvature at the pile head possess always the same sign, which is taken here to be negative. Curvature at the interface deviates from the homogeneous case (where it is constant with depth) by a positive quantity that depends primarily on stiffness contrast between the two layers. As evident by comparing the cases for $h_1/d = 2$ and $h_1/d = 4$, head curvature decreases both with increasing stiffness contrast and decreasing interface depth. The opposite trend is observed for $h_1/d = 10$, where the increase in interface bending with increasing G_2/G_1, leads to an increase in bending at the pile head.
This phenomenon may be interpreted by means of a simple Winkler beam, loaded at one end by a bending moment and fixed against rotation at the other end, as shown in Fig. 5.
On the basis of this scheme, Fig. 5 depicts the restraint reaction, as function of dimensionless beam lenght L/λ_p, where

$$\lambda_p = \sqrt[4]{\frac{4E_p I_p}{k}} \tag{5}$$

is a characteristic wavelength – the reciprocal of the well-known Winkler parameter $\lambda = [k/4E_p I_p]^{1/4}$ (Mylonakis, 1995). Evidently, the pattern is very similar to the distribution of bending along a flexible Winkler beam loaded by a moment at one end (by replacing L/λ_p by x/λ_p on the horizontal axis).
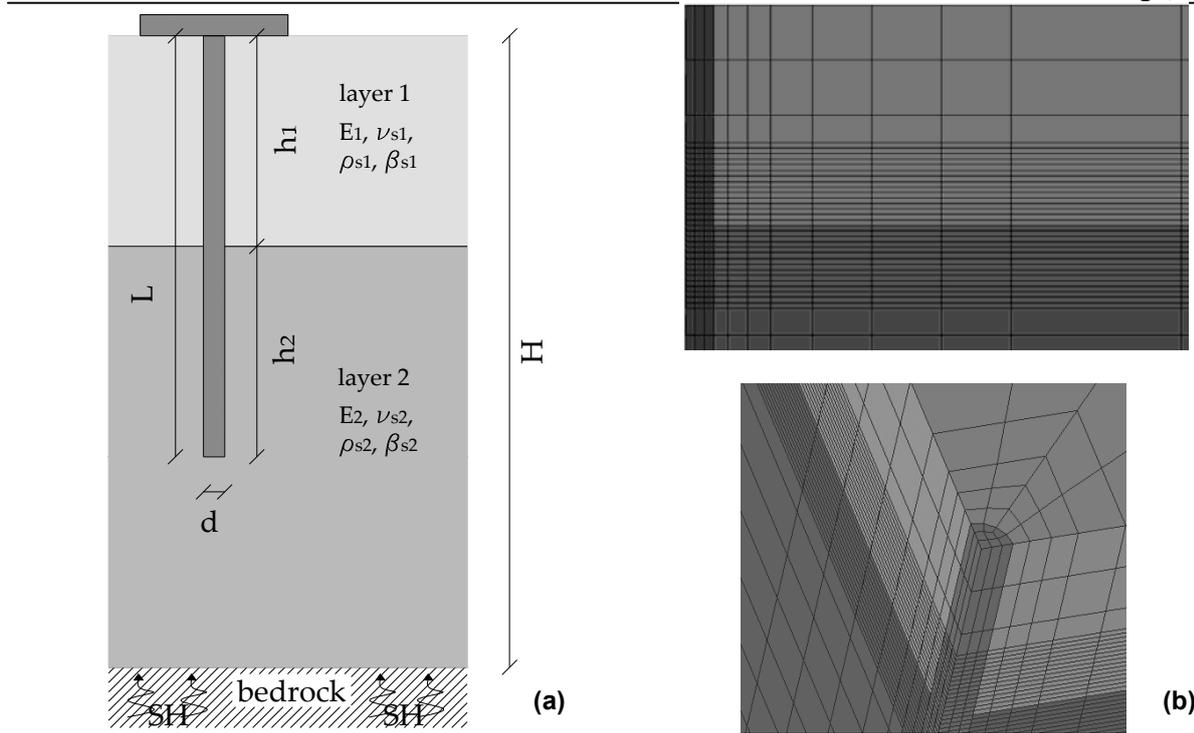
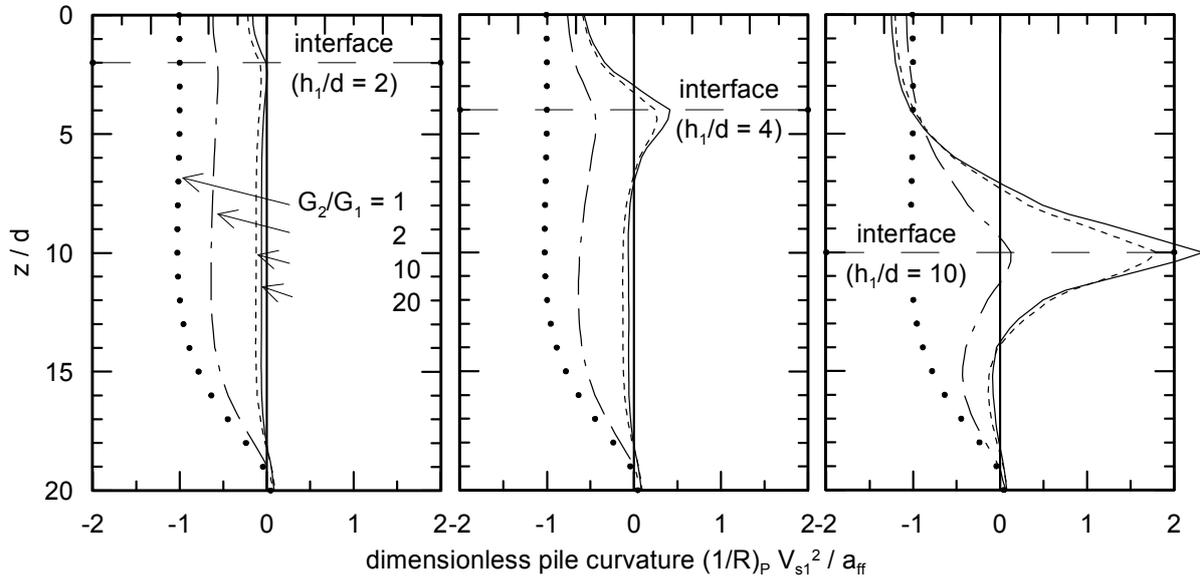**Figure 3. (a) Problem considered: geometry and material properties; (b) Finite - element model**

Figure 4. Pile curvature with depth for static loading ($\omega = 0$), different interface depths (h_1/d) and different layer stiffness contrasts (G_2/G_1); $E_p/E_1 = 1000$.

It can be noticed that up to $L/\lambda_p = 3\pi/4$ the bending moment transmitted to pile cap, R, has the same sign as the forcing moment, M; for longer beams it changes sign, attaining a local maximum at $L/\lambda = \pi$ and then naturally vanishing with increasing beam length.

It is worth noting that this simple scheme should only be interpreted from a qualitative viewpoint, as the actual interaction mechanism is more complicated due to the presence of a certain degree of restraint at the loaded end. Nevertheless, the model is helpful in understanding the physics of bending transmission from the interface to pile head and, hence, exploring the results of Fig. 5.

In the light of these considerations, pile-soil stiffness ratio E_p/E_1 influences λ_p, and, therefore, the critical interface depth where the transmitted moment changes sign. It may be concluded that, while stiffness contrast G_2/G_1 between layers controls the amount of "additional" bending moment at interface, the other two parameters, h_1/d and E_p/E_1, control the fraction of this additional bending moment to be transmitted to the pile head. Evidently, for shallow interfaces, the positive additional bending developing at the interface may reduce the absolute value of bending both at the pile head and the interface.
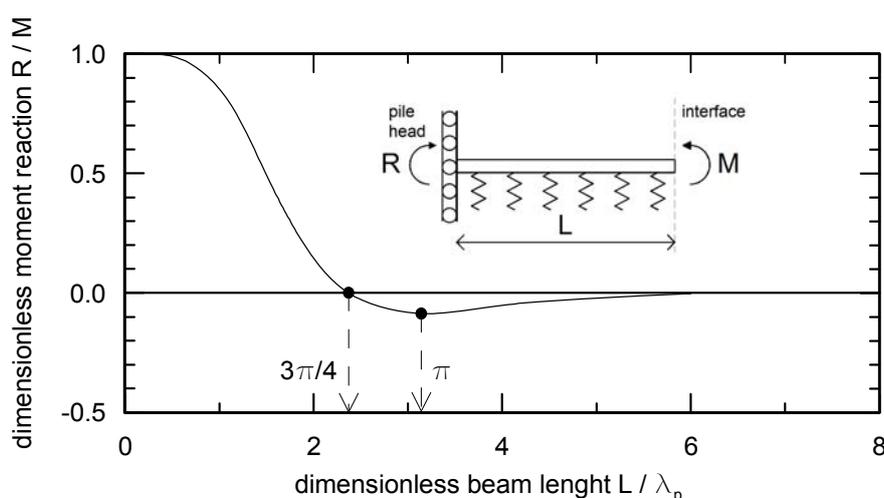

Figure 5. Restraint reaction of a Winkler beam, fixed at one end and loaded at the other end by a concentrated bending moment.

In Fig. 6 pile-soil curvature ratio at pile head is depicted as a function of problem parameters. The mechanisms described above are confirmed by the results shown in the graphs. In Figs 6a and 6b, it is observed that up to a certain interface depth (whose value depends on pile-soil stiffness ratio) an increase in stiffness contrast between the two layers results to a decrease in curvature ratio. On the other hand, beyond a critical interface depth, an increase in h_1/d will decrease curvature ratio with increasing stiffness contrast. Graphs 6c and 6d confirm that pile-soil stiffness ratio plays the same role (through the λ_p parameter) as interface depth. The similarity between the upper and lower graphs in the figure is indeed striking.

FREQUENCY-DOMAIN RESPONSE

Pile-to-soil curvature ratio against dimensionless excitation frequency $\omega d/V_{s1}$ is depicted in Fig. 7. Two issues are worthy of note: *First*, the curvature ratio always decreases with frequency. *Second*, the rate of decrease depends on the parameters described above. As in the homogeneous case, bending clearly decreases because of the difficulty of the pile to respond to short wavelengths. Moreover the rate of decrease with frequency is higher with increasing G_2/G_1, as the latter controls the degree of restraint provided by the second layer. The higher the degree of restraint, the smaller the "free length" of deflection of the pile above the interface. Interestingly, with increasing frequency curvature ratio starts to be independent of the stiffness ratio G_2/G_1, a trend which might be attributed to soil inertia effects. On the

other hand, for a given stiffness contrast (Fig. 7b), the higher the static curvature ratio, the larger the rate of decrease in curvature ratio.
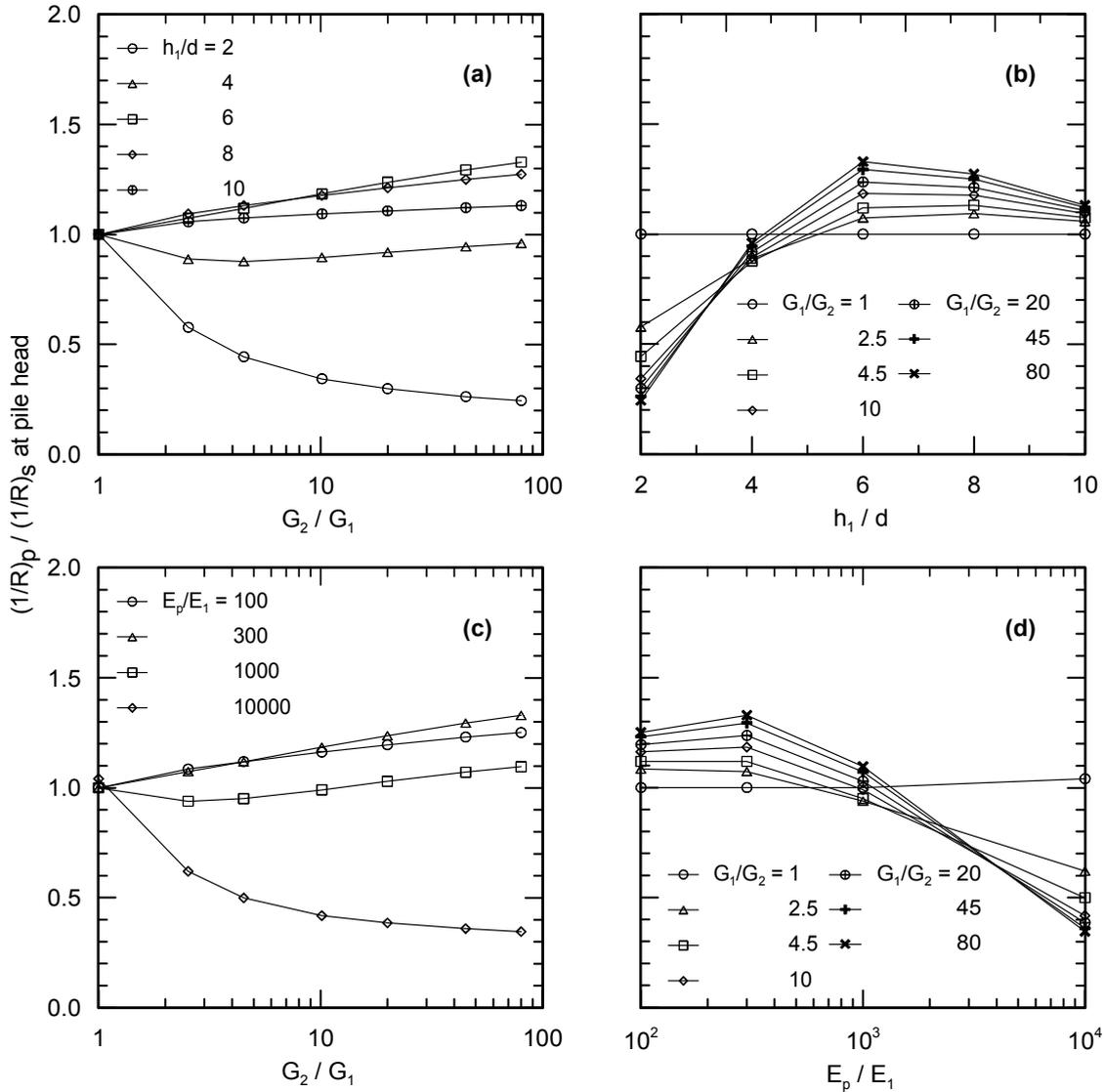

Figure 6. Static curvature ratio. (a) and (b): $E_p/E_1 = 300$; (c) and (d): $h_1/d = 6$.

TIME DOMAIN RESPONSE

As curvature ratio is strongly dependent on frequency, the maximum bending moment at the pile head generated during an earthquake is affected by the frequency content of ground motion. To explore the possibility of developing a simple criterion for estimating kinematic moments at the pile head, parametric analyses were carried out by varying key parameters influencing pile-soil interaction.
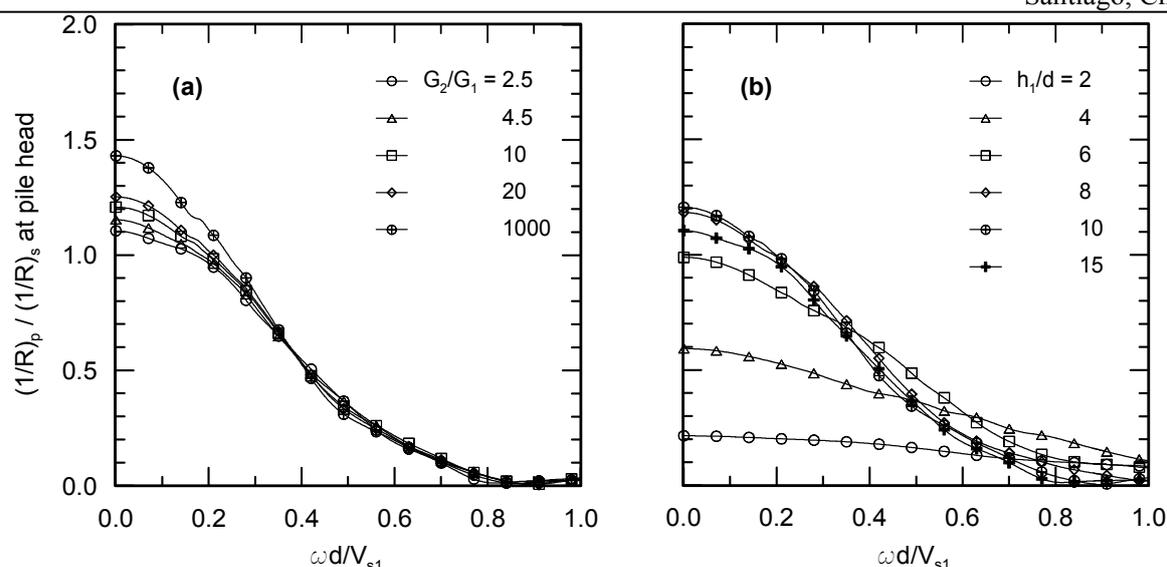
Figure 7. Curvature ratio vs. dimensionless frequency: (a) E_p/E_1 = 1000, h_1/d = 10; (b) E_p/E_1 = 1000, G_2/G_1 = 10.

By applying Buckingham theorem and selecting relevant non-dimensional products that may affect the response, a large number of parameter combinations has to be considered. To limit the number of cases, two different parametric studies were performed by varying only a limited number of dimensionless parameters. Specifically:

In the first parametric study (#1), the fixed parameters were:

H = 30 m; d = 1 m; E_1 = 50 MPa; ρ_p = 2.5 Mg/m^3; ρ_{s1} = 1.6 Mg/m^3; ρ_{s2} = 1.8 Mg/m^3 or 2.0 Mg/m^3; β_p = 0.1; β_{s1} = 0.1; β_{s2} = 0.1; v_p = 0.2; v_{s1} = 0.3; v_{s2} = 0.3; whereas the variable parameters: E_p/E_1 = 300, 1000, 10000; V_{s2}/V_{s1} = 1.5, 2, 3, 6; h_1/d = 2, 4, 8, 16

In the second parametric study (#2), the fixed parameters were:

H = 30 m; d = 0.5 m; E_p = 30 GPa; ρ_p = 2.5 Mg/m^3; ρ_{s1} = 1.6 Mg/m^3; ρ_{s2} = 1.8 Mg/m^3; β_p = 0.1; β_{s1} = 0.1; β_{s2} = 0.1; v_p = 0.2; v_{s1} = 0.3; v_{s2} = 0.3; whereas the variable parameters: E_p/E_1 = 150, 666, 1500; V_{s2}/V_{s1} = 1.5, 2, 3; h_1/d = 4, 8, 16; L/d = 24, 40.

It should be noted that the density of the second layer was taken equal to 1.8 Mg/m^3 when the stiffness contrast is 1.5 or 2, and 2 Mg/m^3 when the stiffness contrast was 3 or 6. Note that in the first study, soil Young's modulus E_1 is constant and pile Young's modulus is variable, whereas in the second study pile Young's modulus is constant and E_1 is variable This difference is by no means coincidental. It is straightforward to show that in the static case (ω = 0) the aforementioned approaches are equivalent. In the dynamic case, however, the two cases are no longer equivalent because of the difference in the dimensionless factor $\omega d/V_{s1}$.

Six signals selected from the Italian Database (Scasserra et al, 2006), covering a wide range of predominant frequencies, were adopted as input motions. The main features of these motions are reported in Table 1; time histories and response spectra in terms of acceleration and pseudo-velocity are provided in Figs 8 and 9. A total of 612 cases was analyzed.

Results

As evident from the previous graphs, the curvature ratio atop the pile is sensitive to the moment generated at the interface. If the latter is located beyond a certain depth (depending on pile-soil stiffness ratio), the interface effect is negligible. For this reason, Fig. 10a shows the correlation between the maximum pile and soil curvatures only for cases in which interface depth is larger than an active length defined as (Gazetas, 1991; Mylonakis, 1995):

$$L_a = 2d \left(\frac{E_p}{E_l} \right)^{0.25} \tag{6}$$

The graph clearly shows that for deep interfaces (a common geotechnical situation) the curvature ratio for kinematic loading can be confidently assumed to be equal to 1. This is a useful and strong result. *First*, no interaction analysis has to be performed to achieve a good estimate of kinematic bending moment at the pile head. *Second*, a ground response analysis is not required either, for soil curvature in homogeneous soil is:

$$\left(\frac{1}{R} \right)_S = \frac{d\gamma}{dz} = \frac{a_s}{V_{s1}^2} \tag{7}$$

$d\gamma/dz$ and a_s being the derivative of shear strain with depth and the acceleration at the soil surface. The value of surface acceleration may be established on the basis of the design Code, thus allowing a simple estimation of kinematic moments without use of numerical tools.

The bending moment at the pile head can be easily calculated as:

$$M_{head} = \left(\frac{1}{R} \right)_p = E_p I_p \frac{a_s}{V_{s1}^2} \tag{8}$$

As an example, a concrete pile with $d = 0.8$ m, $f_{ck} = 25$ MPa and $E_p = 25$ GPa, embedded in a soil having $V_s = 100$ m/s, subjected to a surface acceleration of 2.5 m/s^2, will develop a considerable moment at head equal to 2000 kNm ($\mu_d = M / f_{cd} d^3 = 0.28$).

If the interface is close to the pile head, a complex interaction between the pile and the interface will arise, depending on the aforementioned parameters.

Th e above solution coincides with that obtained by a Winkler model for homogeneous soil and low-frequency excitation and is very similar to the results obtained by de Sanctis et al. (2010). However, the (near unity) numerical factors employed by these authors are not part of Eq. (8). Discussing the origin of these factors lies beyond the scope of this article.

Table 1. Main features of signals used in the analyses.

| Station name	Earthquake	MI	Ep. distance [km]	Component	PGA [g]	Arias Int. [m/s]
TOLMEZZO - DIGA AMBIESTA	Friuli, 06/05/1976	6.4	20	NS	0.36	0.79
STURNO	Irpinia 1st, 23/11/1980	6.5	30	WE	0.32	1.39
BORGO - CERRETO TORRE	Umbria-Marche (AS), 12/10/1997	5.1	10	WE	0.16	0.06
SAN ROCCO	Friuli (AS), 11/09/1976	5.8	24	NS	0.09	0.03
TARCENTO	Friuli (AS), 11/09/1976	5.5	8	NS	0.21	0.11
NOCERA UMBRA - BISCONTINI	Umbria-Marche (AS), 03/10/1997	5.0	7	NS	0.19	0.16
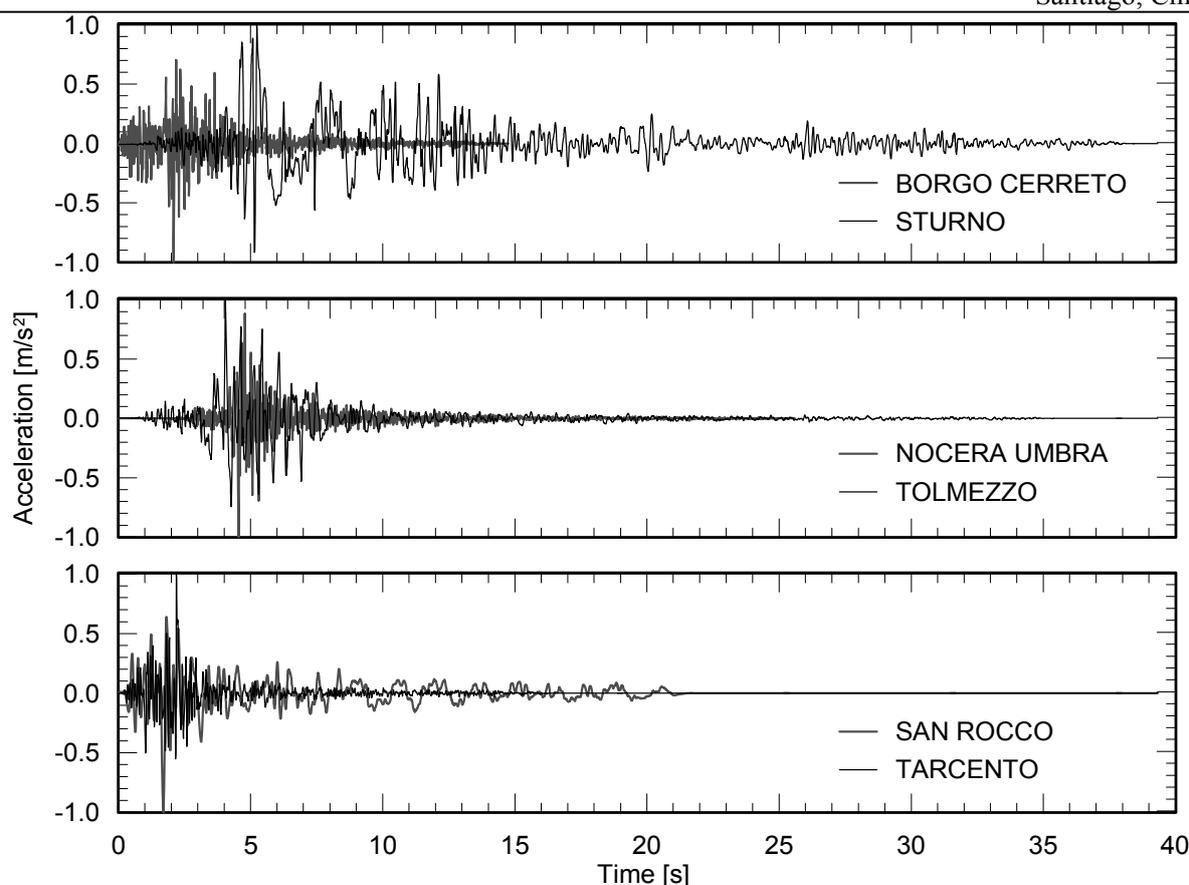
Fig. 8. Acceleration time histories used in the analyses.

Figure 10b shows the aforementioned correlation with reference to a shallow interface ($h_1 < L_a$). It should be noticed that some cases belonging to this group have a curvature ratio that is very close to unity. This clearly reinforces the aforementioned concepts about the influence of a soil layer interface on bending of the pile head. Although at present no simple correlation seems to exist for these cases, it appears that a unit curvature ratio, expressed through Eqs 7 and 8, represents a conservative choice.

CONCLUDING REMARKS

A number of issues related to seismic response of pile foundations have been discussed in light of numerical results regarding piles embedded in layered soil deposits.
Frequency-domain analyses have demonstrated that pile-soil curvature ratio at the pile head may assume values different from unity. As earthquakes are characterised by a large number of frequencies, time-domain results show that the assumption of pile-soil curvature ratio at the pile head equal to 1 provides a satisfactory estimate of bending moments for deep interfaces. For shallow interfaces this assumption generally represents a conservative choice.
It follows that for a given pile-soil configurations, knowledge of a PGA in the free-field may be sufficient to establish the kinematic bending at the pile head.
Kinematic bending may be severe for soft soils and large pile diameters. It is fair to mention that the results and considerations expressed in this paper are limited by the assumption of linear elastic behaviour in the soil and the pile. More research is needed to address these issues.
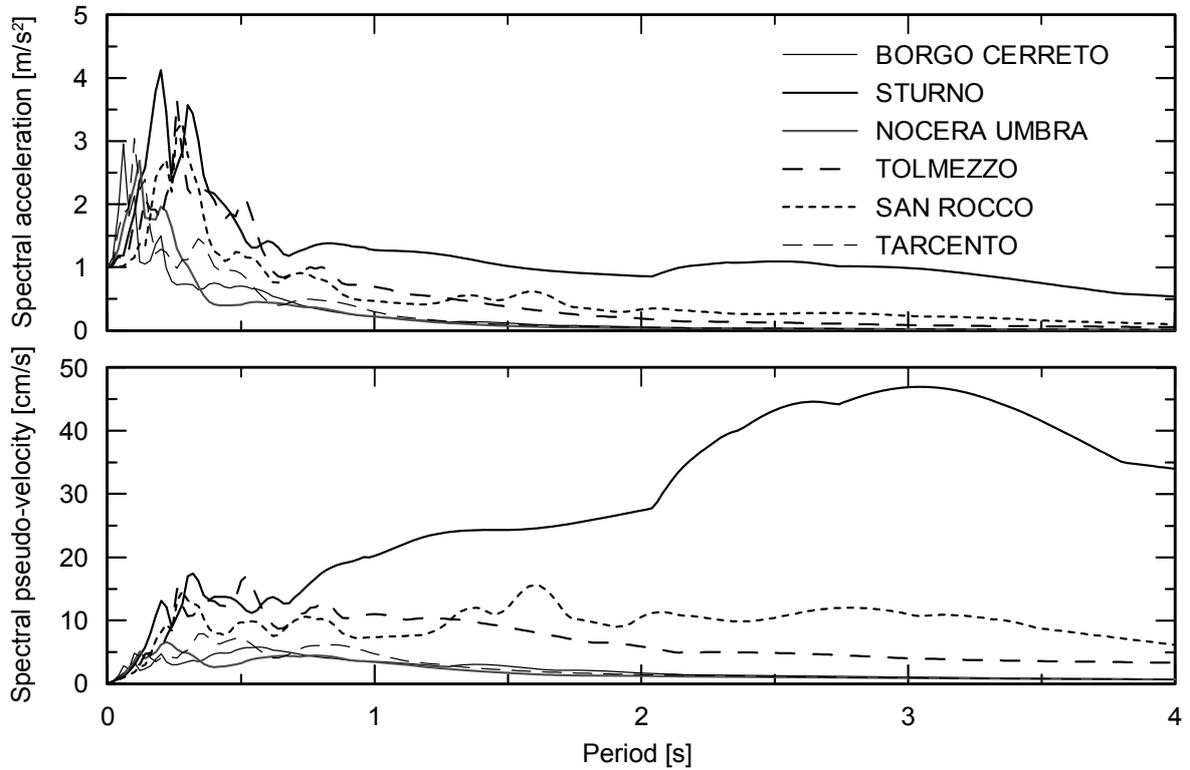
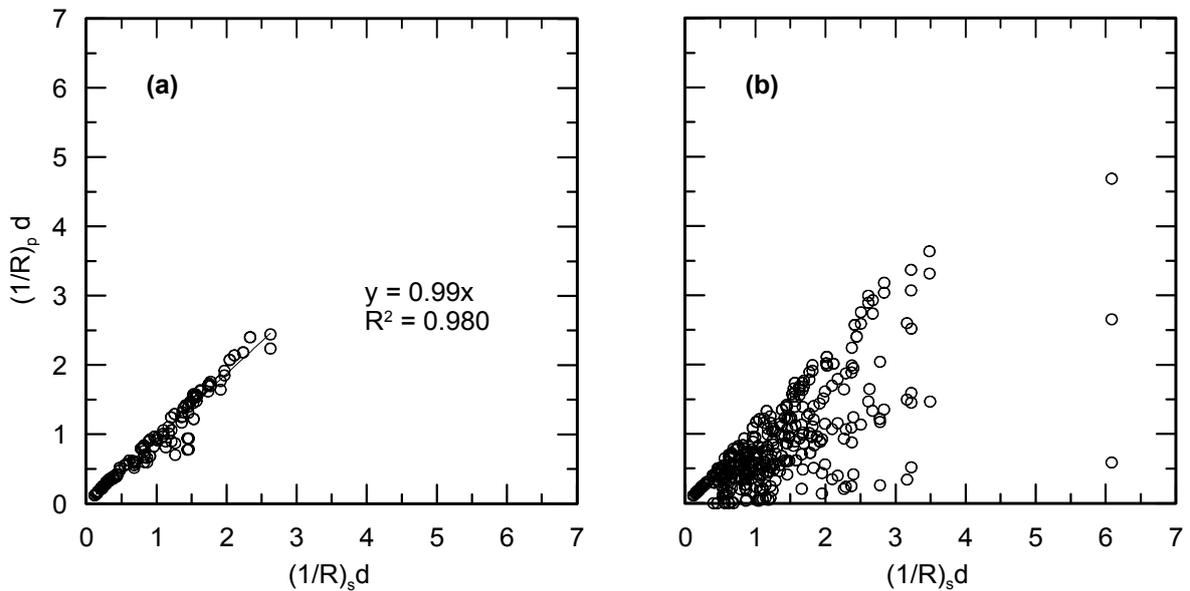**Fig. 9. Response spectra of time histories used in the analyses in terms of acceleration and pseudo-velocity.**

Fig. 10. Pile vs. soil curvature. (a) $h_1 > L_a$**; (b)** $h_1 < L_a$**.**

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