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Pseudostatic analysis of pile groups under earthquake loading

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ABSTRACT: The paper presents a pseudostatic approach for estimating the deformation behaviour and internal forces of single piles and pile groups subjected to seismic excitation. The method is capable of accounting for both inertial and kinematic effects, as required by Eurocode 8. The approach involves two main steps: (1) a free-field site response analysis is performed to obtain the maximum ground displacement profile caused by the earthquake; (2) a static analysis is carried out for the pile group, subjected to the maximum free-field ground displacement profile (kinematic loading) and to the static loading at the pile head based on the maximum surface acceleration (inertial loading). Validity of the approach is illustrated through comparison with alternative numerical analyses. The results indicate that the method has promise in practical applications, offering a reasonable compromise between the uncertainty and limitations of Winkler models and the complexity and time-consuming nature of rigorous dynamic analyses.

1. Introduction

Despite the complexity of the problem of dynamic pile-soil interaction, seismic design of pile foundations is routinely based on pseudostatic approaches which only consider the inertial forces at the pile head generated from the oscillation of the superstructure. The effects of kinematic forces, i.e. forces acting along the length of the pile caused by the passage of seismic waves through the surrounding soil, are generally neglected.

However, recent post-earthquake field investigations have demonstrated the significant role of kinematic effects in the development of pile damage. Following a study of about thirty cases involving seismic failures of piles in Japan, Mizuno (1987) documented a number of pile flexural failures at locations which were too deep to be caused by loading from the pile head (due to structural inertia), while liquefaction could not possibly have occurred. Damage was instead associated with the presence of strong discontinuities in strength and, especially, stiffness of the soil profile. The most likely cause was the relatively large curvature imposed by the surrounding soil as it deforms while excited by up and down propagating seismic waves. The curvatures imposed to the piles by the vibrating soil in turn generate bending moments; these moments will develop even in the absence of a substructure and are referred to as "*kinematic*" moments, to be distinguished from moments generated by lateral loads at the pile head ("*inertial*" moments). A comprehensive set of field records presented by Tazoh *et al.* (1988) and analysed by Nikolaou *et al.* (2001) confirmed the significance of kinematic effects on piles.

The importance of kinematic effects has been recently recognized by seismic regulations such as Eurocode 8 (EN 1998-5, 2003) and the new Italian code (NTC DM 14/01/2008). For example, Part 5 of 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, S_1 or S_2, 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*".

While there is ample geotechnical experience on carrying out equivalent static analyses for the inertial loading [type (*a*)], no specific method or procedure is proposed in current codes to evaluate pile deformations, shear forces and bending moments from the kinematic loading [type (*b*)]. Although a number of analysis methods is currently available, ranging from simplified approaches to sophisticated 3D dynamic boundary element (BEM) or finite element (FEM) formulations, there is a need to better understand the kinematic interaction effect and to develop efficient methods for predicting the pile behaviour in seismic conditions. In this paper, a relatively simple pseudostatic approach is evaluated, which appears to provide reasonable predictions of single pile and pile-group response with little computational effort. The proposed methodology, based on a substructure technique, is capable of accounting for both inertial and kinematic effects.

2. Overview of analysis methods

Before proceeding to a review of analysis methods for seismic design of piles, it is noted that, for both computational convenience and conceptual simplicity, the response of the complete soil-pile-superstructure system is generally computed using a substructure technique based on the superposition of kinematic and inertial response (Gazetas & Mylonakis, 1998). This can be achieved by following three interrelated analysis steps: (1) a free-field site response analysis is carried out to evaluate the response of the soil mass (in the absence of the piles) under seismic excitation (commonly assumed to consist of vertically propagating SH waves); (2) a kinematic analysis is performed to assess the response of the piled foundation to the free-field incoming motion in the absence of inertial forces from the superstructure; (3) an inertial soil-structure interaction analysis is carried out to evaluate the dynamic response of the superstructure and the loads that this response imposes on the foundation.

The above decomposition of the problem does not necessarily imply that the three steps must be performed separately, although this is most often the case in practice. *Complete* interaction analysis (frequently named *direct* analysis) is, at least in principle, also possible. However, with foundations generally consisting of a group of piles, the complexity and computational cost of such analyses become prohibitive for design, particularly when the effects of soil nonlinearity under seismic excitation become significant. It is noted that, from the *superposition theorem* (Kausel & Roesset, 1974), the decomposition into kinematic and inertial response is strictly valid for linear material behaviour (of soil, pile, and structure). However, as an engineering approximation, the superposition may be applied to moderately non-linear systems. This is because pile deformations due to lateral loads from the superstructure inertia attenuate very rapidly with depth (they practically vanish below the so-called *active* pile length, which is typically of the order of 10 to 15 pile diameters below the ground surface). By contrast, the kinematic effects due to free-field input motions are normally important only at relatively large depths. Thus, with soil strains controlled by inertial effects near the ground surface and by kinematic effects at greater depths, the superposition is often a reasonable assumption even when non-linear soil behaviour is expected.

The methods of analysis for estimating the deformation behaviour and internal forces of pile foundations under seismic excitation are mainly based on numerical approaches. These may be broadly classified into the following two categories: (1) Winkler-type (or load-transfer) approaches, and (2) continuum-based approaches, as described below.

2.1 Winkler-type model

This category, initiated by Novak (1974), is based on the so-called beam-on-dynamic-Winkler-foundation (BDWF) approach, in which the pile-soil interaction is simulated through a series of continuously distributed *springs* and *dashpots*, the frequency-dependent parameters of which (the dynamic stiffness "*k*" and the system damping "*c*") have been generally derived through calibration against results of rigorous continuum-based (FEM or BEM) dynamic analyses. This approach has been used extensively to estimate the dynamic impedance (i.e. [$k + i\omega c$], where ω is the loading frequency and *i* denotes an imaginary part) of piles in relation to inertial interaction analyses, i.e. for loads applied at the pile head (Novak, 1991). A number of studies by Gazetas and his co-workers has also employed the Winkler-type model to determine the kinematic response of piles (Kavvadas & Gazetas, 1993; Mylonakis *et al.*, 1997; Nikolaou *et al.*, 2001). In such studies, the springs and dashpots connect the pile to the free-field soil, with the wave-induced motion of the latter (computed with any available method, such as Schnabel *et al.*, 1972) serving as the support excitation of the pile-soil system (Fig. 1).

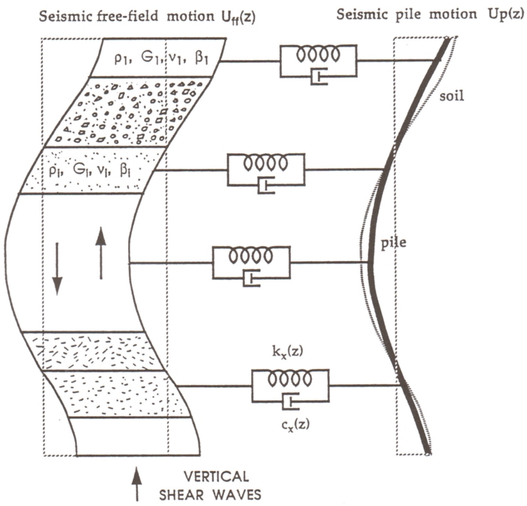

Fig. 1. The Beam-on-Dynamic-Winkler-Foundation (BDWF) model

Based on the above methodology, a number of closed-form expressions for estimating kinematic pile bending moment at the interface between two soil layers has been derived (e.g. Dobry & O'Rourke, 1983; Nikolaou *et al.,* 2001; Mylonakis, 2001). However, one should be aware of the many limitations associated with these simple formulae, such as the general overconservatism, the lack of any information on the pile-head moment (which is additional to the inertial pile-head moment), the limitation to a maximum of two soil layers, and the common assumption of "*thick*" soil layers (i.e. layers with thickness greater than the active pile length). Moreover, the above simplified expressions calculate a value of kinematic bending moment which is directly proportional to the maximum free-field ground acceleration, which is not the case in reality. In fact, significant differences in the values of maximum bending moment may arise under different earthquakes for the same piles in identical soil conditions, even though the earthquake records have the same peak ground acceleration. It is indeed impossible to describe earthquakes by a single parameter such as the peak acceleration (which is only in part a measure of the force involved in the shaking), and other parameters, such as the time history and the frequency content of the ground motion, also play a critical role.

It should also be emphasised that the aforementioned Winkler-type approaches are based on the assumption of linear elastic soil behaviour. However, such an assumption is rather crude for the modelling of the soil, particularly when the inertial loads from the superstructure are to be taken into account. Thus, the linear Winkler models have been extended to deal with soil nonlinearity by making use of the well-known *p-y* curve approach in which the pile inertial effects are modelled by lumped masses and the radiation damping is accounted for by viscous damping (Penzien, 1970; Kagawa & Kraft, 1981; Nogami *et al.*, 1992; El Naggar & Novak, 1996).

Turning to pile group response, it is widely recognized that pile-to-pile interaction is of basic importance in evaluating the inertial effects (i.e. head-loaded piles), while its effect is generally less significant for the kinematic response. In assessing inertial effects, the group impedances are usually determined by making use of Poulos' static superposition procedure, extended to dynamic loading by Kaynia & Kausel (1982) and Gazetas *et al.* (1993). The approach makes use of frequency-dependent interaction factors (i.e. displacement ratios expressing the influence of one pile onto another) which have been derived by matching the dynamic pile-head displacements of the Winkler approach with rigorous finite element analyses.

Although Winkler models (such as the *p-y* curve approach) have become popular for the seismic analysis of pile foundations, mainly due to their relative simplicity, one should be aware of the following assumptions and limitations associated with the approach:

(a) Single-pile response:

The Winkler model is of semi-empirical nature in that the spring coefficient is not a fundamental soil parameter but instead gives the overall effect of the soil continuum as seen by the pile at a specific depth and hence its value depends not only on the soil properties but also on the pile dimensions. Thus, no direct soil tests can be conducted to establish the spring coefficient of the load-transfer curves for that particular pile and soil type, and hence standard curves are usually adopted in engineering practice. However, there are many uncertainties in such a procedure and the difficulties in estimating the spring stiffness are well-known (e.g. Poulos *et al.*, 2001; Basile, 2003; Finn, 2005). Two major studies in the mid-eighties (Murchison & O'Neill, 1984; Gazioglu & O'Neill, 1984), involving data from 35 monotonic and 19 cyclic loading full-scale tests of piles, concluded that the *p-y* constitutive model gives poor predictions and is fairly unreliable. In addition, the model has been little calibrated for seismic loading conditions and cannot be expected to perform better in the seismic environment. Thus, in the aforementioned studies of Gazetas and other researchers, the stiffness and damping coefficients of the Winkler model have usually been derived through curve-fitting, i.e. by matching the results of rigorous continuum-based (FEM or BEM) dynamic approaches. The validity of such an approach for general pile, soil, and loading conditions is uncertain.

It is interesting to note that, in the evaluation of the kinematic response, the value of the stiffness coefficient has a relatively small influence on the maximum pile moment (the stiffness contrast between layers has a more significant role), and this may explain the relative success of the method. However, in evaluating the response of head-loaded piles (inertial effects), the stiffness coefficient plays a dominant role and hence the difficulties in selecting an appropriate value become apparent.

(b) Group effects:

The Winkler approach treats the soil as a series of springs which are independent and do not interact, i.e. the displacement of one spring has no effect on the displacement of any other springs. This neglects continuity through the soil and makes it impossible to find a rational way to quantify the interaction effects between piles in a group. Thus, in evaluating inertial group effects, recourse is usually made to an extension of Poulos' static superposition approach to the dynamic case. However, the superposition of two-pile interaction factors is an approximate procedure which produces a number of limitations, such as it ignores the stiffening effect of intervening piles in a group, its use is questionable for dissimilar piles, and the calculated distribution of loads and moments along piles is only approximate. Although some of these deficiencies appear to have been addressed in the static case (e.g. Randolph, 2003), the superposition approach remains an approximate procedure, particularly in the dynamic environment where little calibration work has been carried out. With regard to kinematic group effects, these are usually ignored by Winkler models, despite the fact that some researchers have shown that such effects are small but not insignificant (Nikolaou *et al*., 2001; Dezi *et al*., 2009).

(c) *Load-deformation coupling:*

Pile-soil interaction is a three-dimensional problem and each of the load components has deformation-coupling effects. For example, a lateral load acting on a group of piles will also generate axial loads (as well as lateral loads) on the piles to counteract rocking of the pile group. This aspect, which is particularly important in real design (where the pile group is subjected to a simultaneous combination of axial and lateral forces), cannot be analysed by the Winkler model.

2.2 Continuum-based approach

The main limitations of Winkler models may be removed by means of rigorous continuum-based dynamic solutions, generally based on the finite element or the boundary element method (e.g. Kania & Kausel, 1982; Sen *et al*., 1985; Mamoon, 1990; Maheshwari *et al*., 2004). These solutions provide an efficient means of retaining the essential aspects of pile interaction through the soil continuum and hence a more realistic representation of the problem. However, as noted by Tabesh & Poulos (2007), some of these solutions are mathematically cumbersome and very complex to use (for example Tabesh & Poulos (1997) found that not all of these methods yield identical results, even when the fundamental assumptions are the same). In addition, such analyses are limited by the high computational costs which may be justified only for research purposes or for very large projects.

In fact, the disturbance travels as a wave in the ground and, contrary to the static case, where the influence of the load is confined to a limited area around the application point of the load, a very large area is affected. Thus, a FEM mesh generally needs to be very large to accommodate radiation damping and very dense to allow correct representation of prominent frequencies in the ground motion. This makes conventional FEM analyses inefficient for dynamic analysis. On the other hand, dynamic BEM analyses have the ability to automatically satisfy the condition of radiation damping. However, they are mathematically more complicated and by nature far less flexible than FEM analyses, particularly in modelling soil nonlinearity and soil nonhomogeneity. In addition, even though the 3D problem may be reduced to a soil-pile interface problem, dynamic BEM analyses remain time-consuming and complex to use for routine design.

In order to overcome the shortcomings of dynamic analyses, a more practical approach has recently been proposed by Tabesh & Poulos (2001) for single piles in linear elastic soil. The approach is based on a pseudostatic procedure involving two subsequent steps: computation of the soil movements via a free-field seismic analysis and then, by means of a static boundary element analysis, computation of the pile response subjected to the computed free-field soil movements (kinematic loading), in addition to the static loading at the pile head (inertial loading). On the basis of numerous comparisons with rigorous dynamic analyses, the above work demonstrated that, when the pile response is governed by the free-field ground movements (this is the case if the cap-mass is not large enough to bring the natural period of the pile-cap-soil system within the range of dominant periods of the surface motion), the static interaction between pile and soil plays a dominant role, and an excellent agreement between the pseudostatic and dynamic analyses is observed. With the increase in cap-mass, the agreement between the pseudostatic and dynamic analyses is in some cases reduced, with a tendency of the pseudostatic approach to overestimate the pile internal forces by up to 25% (which is an acceptable conservatism for practical purposes). One reason is that, in the pseudostatic analysis, the maximum free-field effects and the maximum inertial effects have been assumed to act simultaneously (i.e. in phase), which does not occur in a dynamic analysis. One of the advantages of the pseudostatic procedure is that, by ignoring the generated waves at the pile-soil interface, it greatly simplifies the problem and makes any effort for modelling radiation damping unnecessary. In fact, in dynamic analyses, the generated interaction waves need to be damped out from the model in order to simulate actual conditions (where the waves travel outwards towards infinity and are not reflected back to the pile-soil interface). The pseudostatic methodology by Tabesh & Poulos (2001) has also been employed to analyse the case of piles in liquefying soil (Liyanapathirana & Poulos, 2005) and, despite its simplicity, a good agreement with the results from dynamic analyses is confirmed.

3. Proposed method of analysis

The pseudostatic approach presented in this paper is similar to that proposed by Tabesh & Poulos (2001) for single piles in linear elastic soil, which is extended to include the effects of group interaction and soil nonlinearity (via a hyperbolic continuum-based soil model). The numerical procedure is carried out within PGROUPN (Basile, 2003, 2010), a completely general computer program for determining the axial, lateral, rocking, and torsional response of pile groups by means of a boundary element formulation. The work makes use of Mindlin solution (1936) to perform a "complete" analysis of the group (i.e. the simultaneous influence of all the elements of all the piles within the group is considered), thereby removing the approximations of the interaction factor approach employed by Winkler models. The program has negligible computational costs and is widely used in pile group design through the commercial software Repute (Bond & Basile, 2009).

The proposed pseudostatic approach, based on the substructure technique, is capable of accounting for both inertial and kinematic effects, and involves two main steps:

(1) Free-field site response analysis in order to obtain the maximum ground displacement profile along the pile and the maximum ground surface acceleration generated by the earthquake;

(2) Static BEM analysis of the pile group, subjected to the maximum free-field ground displacement profile along the pile (kinematic loading) and to the pile cap load given by the cap-mass multiplied by the maximum free-field ground surface acceleration (inertial loading).

3.1 Free-field site response

By assuming that the earthquake consists of vertically incident SH waves, a free-field site response analysis is performed to obtain both the maximum ground displacement profile along the pile (specifically at the pile nodes) and the maximum ground surface acceleration during the earthquake loading. For this purpose, the well-known SHAKE program (Schnabel *et al.*, 1972) or similar codes such as the EERA program (Bardet *et al.*, 2000) used herein, may be employed. Such codes adopt the concept of wave propagation in a layered medium and model the nonlinearity of the shear modulus and damping ratio by the use of an equivalent linear procedure.

3.2 Pile-group static analysis

The pile group response is determined by means of a static BEM analysis in which the external loads are the computed maximum free-field ground displacement profile along the pile (kinematic loading) and the pile cap load given by the cap-mass (representing the mass of the superstructure) multiplied by the maximum free-field ground surface acceleration (inertial loading). It should be emphasised that the envelope of the maximum free-field ground displacement profile is used, even though the displacement at each pile node may have occurred at different times.

In order to statically apply the computed free-field ground displacements profile to the piles, the PGROUPN analysis, originally developed for direct applied loading at the pile cap level (i.e. inertial-type loading), has been extended to deal with externally imposed ground movements in both the axial and lateral directions. The capability of also applying axial ground movements may be useful in the case of raked piles in which the horizontal ground movements obtained from the free-field analysis can be applied as axial and lateral components. It is noted that the extended PGROUPN analysis may be employed not only in the seismic case described herein but in many circumstances in which pile foundations are subjected to “passive” loadings arising from vertical and/or horizontal movements of the surrounding ground. Examples include slope movement, consolidation of clay, swelling or shrinking of an expansive clay, tunnelling, excavation, cavity development, and construction of adjacent piles or buildings. This represents a significant advance over previous work which is generally restricted to de-coupled passive loadings and cannot analyse cases where vertical and horizontal soil movements act together and influence both the vertical and lateral response of the piles simultaneously.

A description of the theoretical formulation of the PGROUPN analysis for the case of direct applied (“active”) loading has been presented elsewhere (Basile 2003, 2010). The modelling of the pile-soil interaction problem in “active” and “passive” piles is quite similar and hence only a brief description of the passive case is given below. The analysis is based on a complete non-linear BEM formulation and involves discretization of only the pile-soil interface into a number of elements, each element being acted upon by an unknown uniform stress (Fig. 2). The method employs a substructure technique in which the piles and the surrounding soil are considered separately and then compatibility and equilibrium conditions are imposed at the interface.

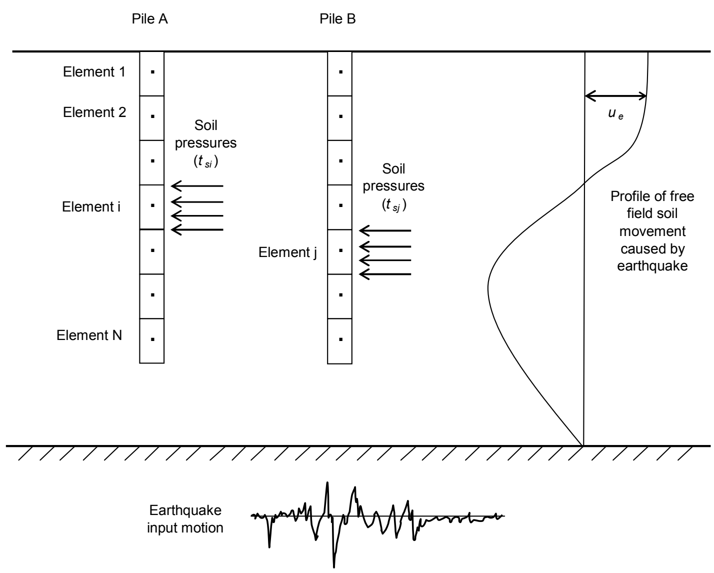

Fig. 2. Pseudostatic BEM schematisation of the problem

3.2.1 Soil domain

The soil displacements, arising both from the stresses caused by pile-soil interaction and the external source of ground movement, may be expressed as:

$$\{u_s\} = [G_s]\{t_S\} + \{u_e\} \qquad \text{(Equ. 1)}$$

where u_s are the soil displacements, t_s are the soil stresses, G_s is the soil flexibility matrix obtained from Mindlin's (1936) solution, and u_e are the external soil movements. It is noted that Mindlin's solution is strictly applicable to homogeneous soil conditions. In practice, however, this limitation is not strictly adhered to, and the influence of soil non-homogeneity is often approximated using the average value of soil modulus at the influencing and influenced pile nodes (Poulos, 1979; Tabesh & Poulos, 2001).

3.2.2 Pile domain

If the piles are assumed to act as simple beam-columns which are fixed at their heads to the pile cap, the pile displacements may be written as:

$$\{u_p\} = \lfloor G_p \rfloor \{t_p\} \qquad \text{(Equ. 2)}$$

where u_p are the pile displacements, t_p are the pile stresses, and G_p is a matrix of coefficients obtained from the elementary (Bernoulli-Euler) beam theory.

3.2.3 Limiting pile-soil stress and non-linear soil behaviour

It is essential to ensure that the stress state at the pile-soil interface does not violate the yield criteria. This can be achieved by specifying the limiting stress at the pile-soil interface using the classical equations (refer to Basile, 2003). Non-linear response of the soil is modelled, in an approximate manner, by assuming that the soil Young's modulus varies with the stress level at the pile-soil interface according to the popular hyperbolic stress-strain law proposed by Duncan & Chang (1970):

$$E_{\tan} = E_i \left(1 - \frac{R_f t}{t_{\lim}}\right)^2 \qquad \text{(Equ. 3)}$$

where E_{tan} is the tangent soil modulus, E_i is the initial tangent soil modulus, R_f is the hyperbolic curve-fitting constant, t is the pile-soil stress and t_{lim} is the limiting value of pile-soil stress. Thus, the soil and pile equations described above for the linear response are solved incrementally using the modified values of soil Young's modulus of Equ. (3) and enforcing the conditions of yield, equilibrium and compatibility at the pile-soil interface.

4. Verification of the method

The performance of the proposed pseudostatic methodology, as implemented in PGROUPN, is assessed through a comparison with alternative numerical procedures, ranging from simplified approaches to rigorous dynamic analyses.

4.1 Comparison with Poulos (2006)

In order to illustrate the crucial difference between the effects of loading induced by external ground movements and direct applied loading, a single free-head pile in a two-layer soil profile is analysed under the following types of (static) loading (Fig. 3):

(a) An axial load of 1.0 MN applied at the pile head;
(b) An axial ground movement profile decreasing from 100mm at the ground surface to zero at a depth of 12m;
(c) A lateral load of 0.1 MN applied at the pile head;
(d) A lateral ground movement profile decreasing from 100mm at the ground surface to zero at a depth of 12m.

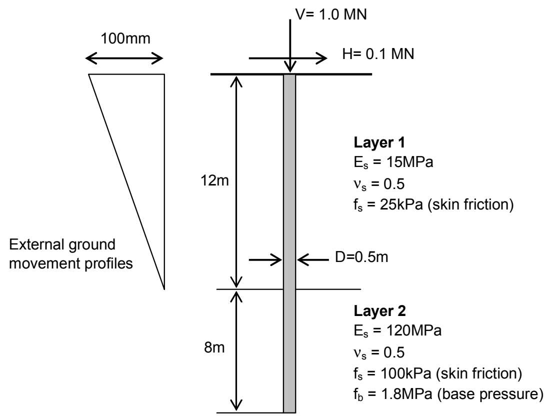

Fig. 3. Free-head pile in two-layer soil system

The resulting axial load and bending moment distributions computed by PGROUPN are displayed in Figs. 4-5, showing a favourable agreement with the results reported by Poulos (2006) using a similar BEM approach (Poulos & Davis, 1980) which employs an elastic-perfectly plastic continuum-based interface model (corresponding to R_f =0 in Equ. 3). The resulting axial load distribution for loading types (a) and (b) is reported in Fig. 4, together with the distribution generated by the two type of loadings (a) and (b) acting together, and the distribution resulting from the addition of the two profiles of axial load obtained for loadings (a) and (b). It is noted that the distribution of axial load in the pile due to direct applied loading is very different from that induced by the ground movements. In the latter case, the maximum axial load occurs near the bottom of the zone subjected to ground movement. Moreover, the simple addition of the two profiles of axial load obtained for loadings (a) and (b) yields axial loads which are less than those arising from the simultaneous application of loadings (a) and (b). This is because the axial ground movement has caused full slip at the pile-soil interface of the upper layer (i.e. the limiting pile-soil skin friction is reached) and hence the superposition principle does not apply.

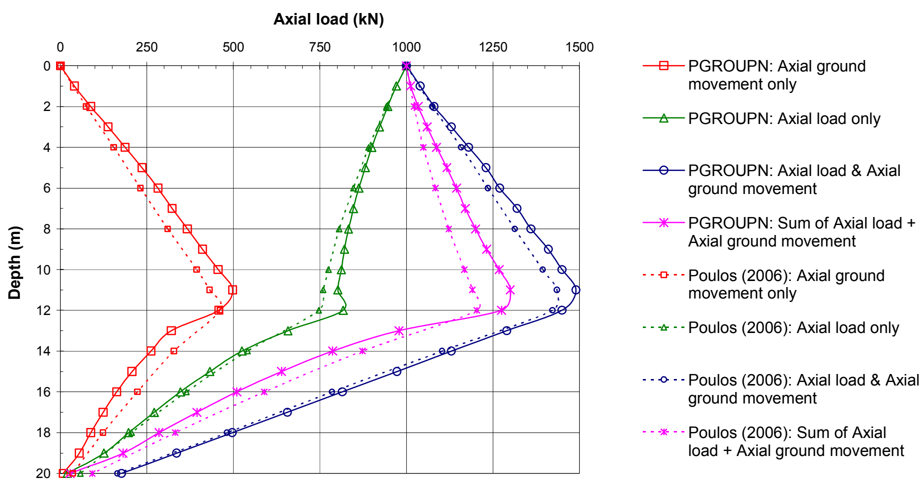

Fig. 4. Comparison of axial distribution with Poulos (2006)

Fig. 5 shows the corresponding bending moment distributions computed for the lateral response of the pile. Again, it is observed that the distribution of induced moment is very different for direct lateral load and for lateral ground movement. In the latter case, the maximum moment occurs well below the pile-head, near the bottom of the upper layer which is subjected to ground movements. The maximum moment under the combined loadings also occurs at the latter location, where the moment due to the lateral load is insignificant and the moment due to the lateral ground movement is largest. It is noted that the simple addition of the two moment profiles obtained for loadings (c) and (d) overlaps with the moment profile caused by the simultaneous application of loadings (c) and (d). This is because no yielding at the pile-soil interface has occurred and hence the superposition principle applies closely.

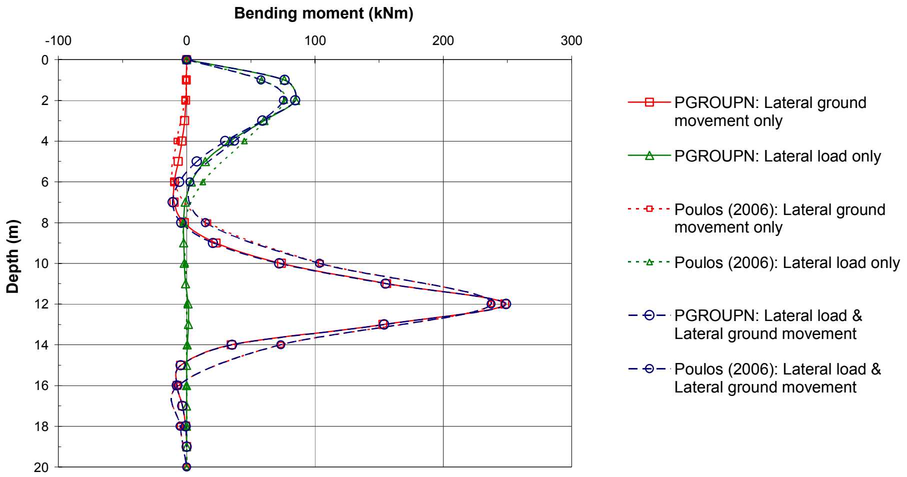

Fig. 5. Comparison of bending moment distribution with Poulos (2006)

The basic difference between the effects of direct applied loading (i.e. inertial-type) and loading induced by external ground movements (i.e. kynematic-type) is further examined through the analysis of a fixed-head single pile in a layered soil profile, as shown in Fig. 6 (ground type C according to Eurocode 8). The loading conditions are a lateral "inertial" load of 0.2 MN at the pile head and/or a "kinematic" ground movement profile derived from an elastic free-field analysis using the EERA code. It is assumed that the site is subjected to the 1994 Northridge earthquake (Sepulveda station, SPV270 record from PEER database, PGA = 0.753g), scaled to a maximum bedrock acceleration of 0.2g.

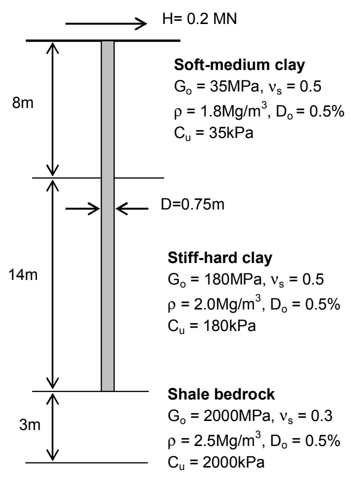

Fig. 6. Fixed-head pile in layered soil system

Under the assumption of linear elastic soil behaviour, the bending moment distributions computed from PGROUPN are reported in Fig. 7. The results compare favourably with those reported by Poulos (2006) using the pseudostatic procedure of Tabesh & Poulos (2001), showing the crucial importance of accounting for both inertial and kinematic effects. If kinematic effects are ignored, and only inertial loading at the pile-head is considered, this results in a considerable underestimation of moment at the pile-head. In addition, the effect of kinematic loading leads to a significant moment at the interface between the soft upper layer and the stiff lower layer. The above results (and many similar findings not presented herein for lack of space) suggest that the evaluation of kinematic bending moments may be important not only for ground types D or worse, as recommended by Eurocode 8, but also for ground type C. For comparison, the kinematic bending moment at the layer interface has been calculated (using the maximum free-field ground acceleration computed by EERA) from the aforementioned expressions by Dobry & O'Rourke (1983), Nikolaou *et al*. (2001), and Mylonakis (2001), yielding values of 298, 288, and 311 kNm, respectively, i.e. an average overestimation by about 50%.

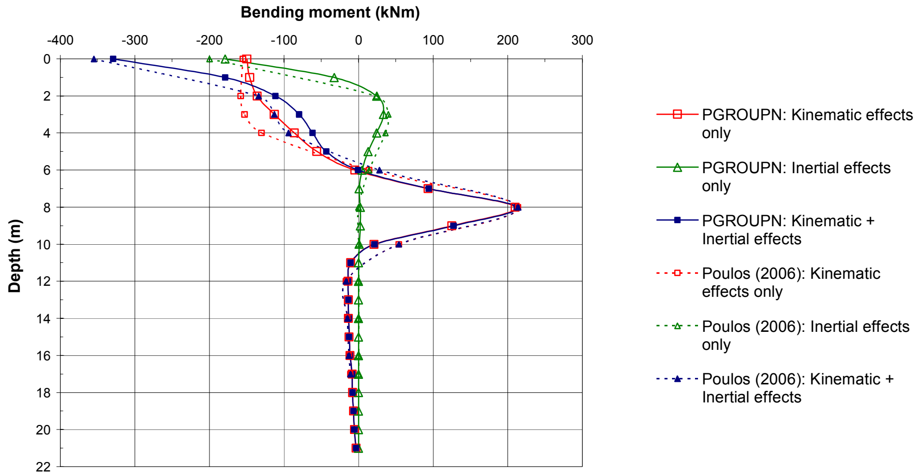

Fig. 7. Comparison of bending moment distribution with Poulos (2006)

Fig. 8 illustrates the effect of soil nonlinearity on bending moment distribution as computed by PGROUPN using the limiting stress and the non-linear model at the pile-soil interface described above. The PGROUPN analysis has been preceded by a non-linear free-field EERA analysis using the default degradation curves for the shear modulus and damping ratio (Seed & Sun, 1989; Idriss, 1990). For comparison, results from two purely elastic PGROUPN analyses (based on the elastic free-field EERA analysis) are shown, one using the same initial ("small-strain") soil moduli E_{so} employed in the non-linear analysis (as already reported in Fig. 7), and the other one using reduced (secant) values of soil modulus (arbitrarily taken as 0.3 times the small-strain values E_{so}) to account for the higher strain levels induced by the earthquake loading (e.g. EC8-Part 5; Poulos *et al*., 2001). For the two values of soil modulus (E_s=E_{so} and E_s=0.3E_{so}), the linear analyses result in a relatively large range of values for the maximum bending moments, varying between 179 and 249 kNm at the pile-head (inertial effects), and between 143 and 210 kNm at the layer interface (kinematic effects). This shows the shortcomings of a purely linear analysis which is limited by the problematic selection of an appropriate secant value of soil modulus (relevant for the actual strain level). By contrast, a non-linear interface model (such as that employed by PGROUPN) gives a more realistic representation of the problem and has the advantage of adopting initial (small-strain) values of soil modulus, which are a more reproducible quantity.

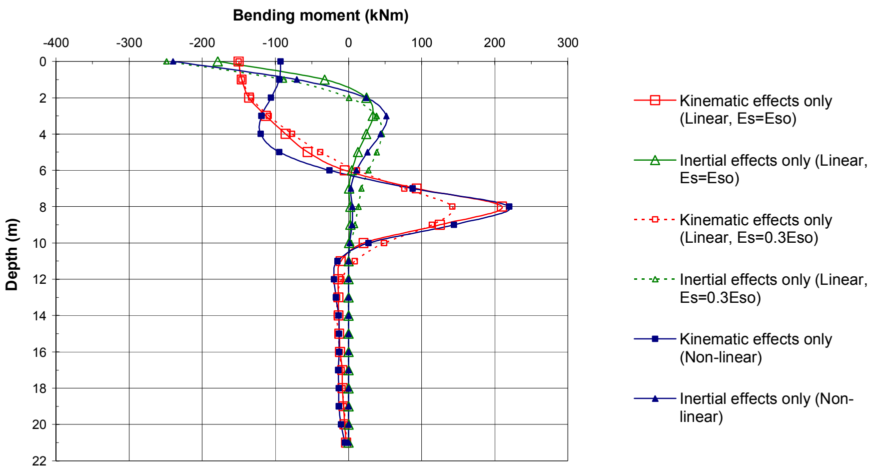

Fig. 8. Influence of soil nonlinearity on bending moment distribution

4.2 Comparison with RELUIS project

The PGROUPN pseudostatic approach is compared with alternative methods (ranging from simplified procedures to rigorous dynamic analyses) for the reference scheme adopted within the RELUIS project, a major research activity carried out by a consortium of Italian universities (e.g. RELUIS, 2007, 2009; Maiorano *et al*., 2009). Under the assumption of linear elastic soil behaviour, the comparisons illustrate the effects of *kinematic* loading on a fixed-head single pile (length L = 20m, diameter D = 0.60m, Young's modulus E_p = 25 GPa) embedded in a two-layer soil profile underlain by rigid bedrock (Fig. 9). The bedrock is located at a fixed depth of 30 m, while the interface between layers is located at variable depths (specifically, H_1 = 5, 10, 15, and 19 m). The acceleration time histories of Table 1 have been selected from the SISMA database of Italian seismic events (Scasserra *et al*., 2008). The input motions have been scaled to a maximum bedrock acceleration of 0.35g and applied to the top of the bedrock (i.e. without considering any deconvolution).

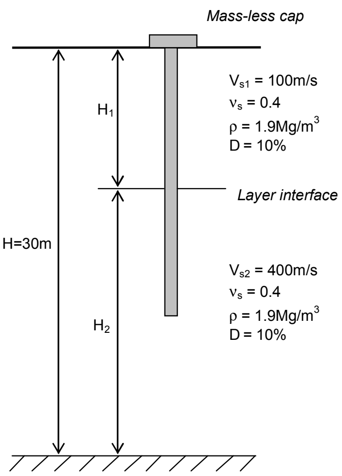

Fig. 9. Reference scheme for comparison with RELUIS project

Table 1. Acceleration time-histories from the SISMA database

Label	Earthquake	Date	Station	M_w	PGA (g)
A-TMZ000	Friuli	06.05.1976	Tolmezzo-Diga Ambiesta	6.5	0.357
A-TMZ270	Friuli	06.05.1976	Tolmezzo-Diga Ambiesta	6.5	0.315
A-STU000	Irpinia	23.11.1980	Sturno	6.9	0.223
A-STU270	Irpinia	23.11.1980	Sturno	6.9	0.321
A-AAL018	Umbria-Marche	26.09.1997	Assisi-Stallone	6.0	0.189
E-NCB090	Umbria-Marche (aftershock)	06.10.1997	Nocera Umbra-Biscontini	5.5	0.383

Fig. 10 compares the maximum kinematic bending moment obtained at the layer interface from different procedures, including the PGROUPN analysis, the simplified closed-form expressions by Dobry & O'Rourke (1983), Nikolaou *et al.*, (2001), and Mylonakis (2001), the beam-on-dynamic-Winkler-foundation (BDWF) approaches by Conte & Dente (1989), Sica *et al*. (2007), and Dezi *et al*. (2007) (as reported by Moccia *et al*., 2009), and the rigorous dynamic BEM analysis by Cairo & Dente (2007) using the SASP code. A number of trends emerges from this figure: (1) kinematically induced bending moments can be important at the layer interface and hence cannot routinely be neglected in design, (2) the simplified expressions tend to overestimate bending moments significantly, and (3) PGROUPN predictions of bending moments are within 19% of those obtained using the rigorous SASP analysis and within 23% of the average results of the BDWF approaches, which is acceptable for practical pile design purposes.

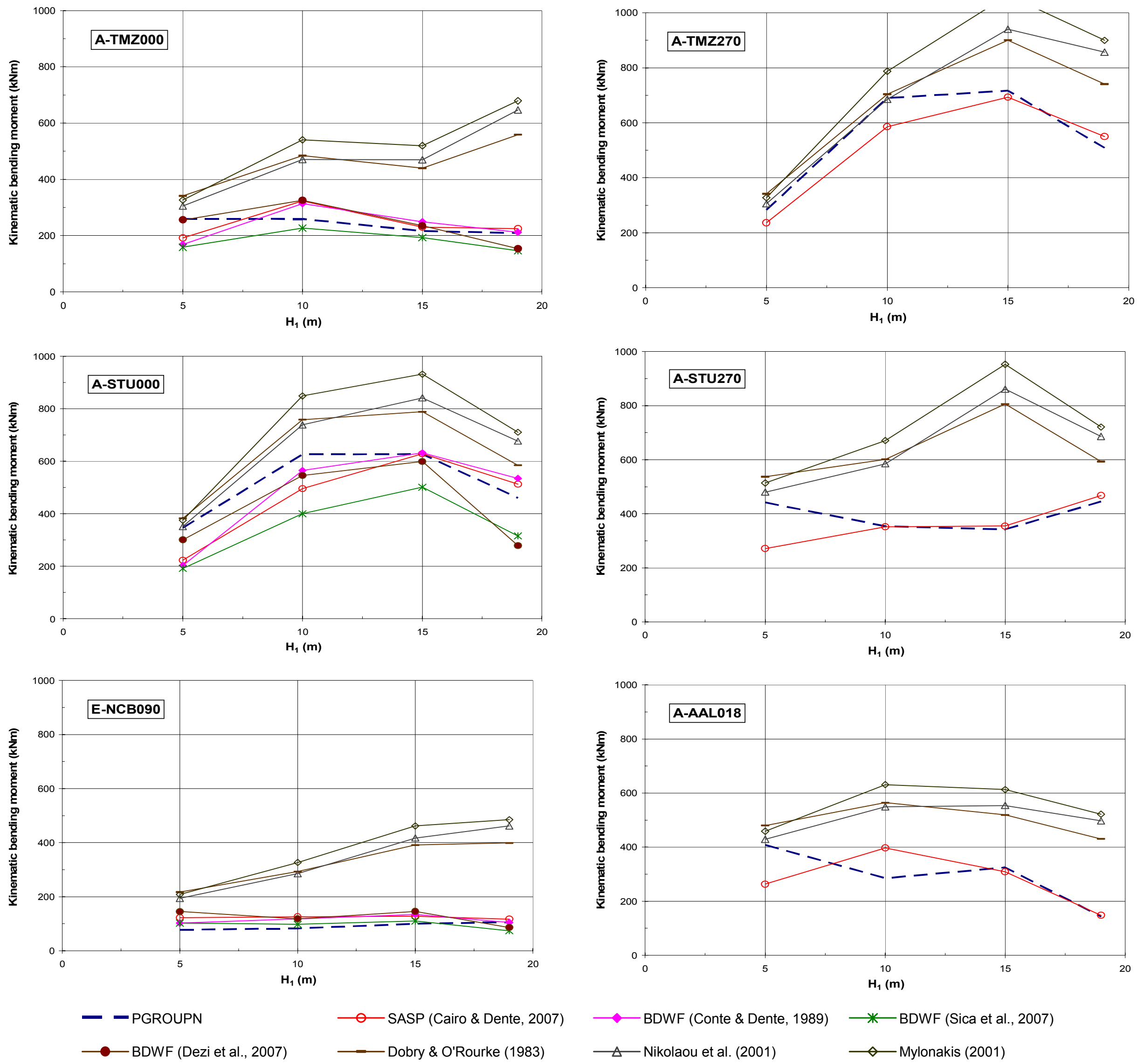

Fig. 10. Comparison of maximum bending moment at layer interface

A comparison of the kinematic bending moment distributions obtained in the case H_1= 15 m is shown in Fig. 11. In order to facilitate the comparison with the moment distribution from the pseudostatic PGROUPN analysis, the envelope of the positive moments calculated from the dynamic SASP and BDWF analyses (RELUIS, 2007) has been mirrored with respect to the pile axis. It is worth noting that the static profile in some parts matches the dynamic positive envelope and, in other parts, the dynamic negative envelope. The good agreement between the static and dynamic analyses suggests that the maximum values of soil displacement along the pile have occurred at the same time step in the free-field analysis (clearly, the static analysis is "blind" as to the direction of the developed moment because the absolute value of the maximum free-field displacements is used). The closeness of the results is not confined to the maximum moment values but occurs along the entire length of the pile, thereby confirming the aforementioned findings by Tabesh & Poulos (2001) and Liyanapathirana & Poulos (2005).

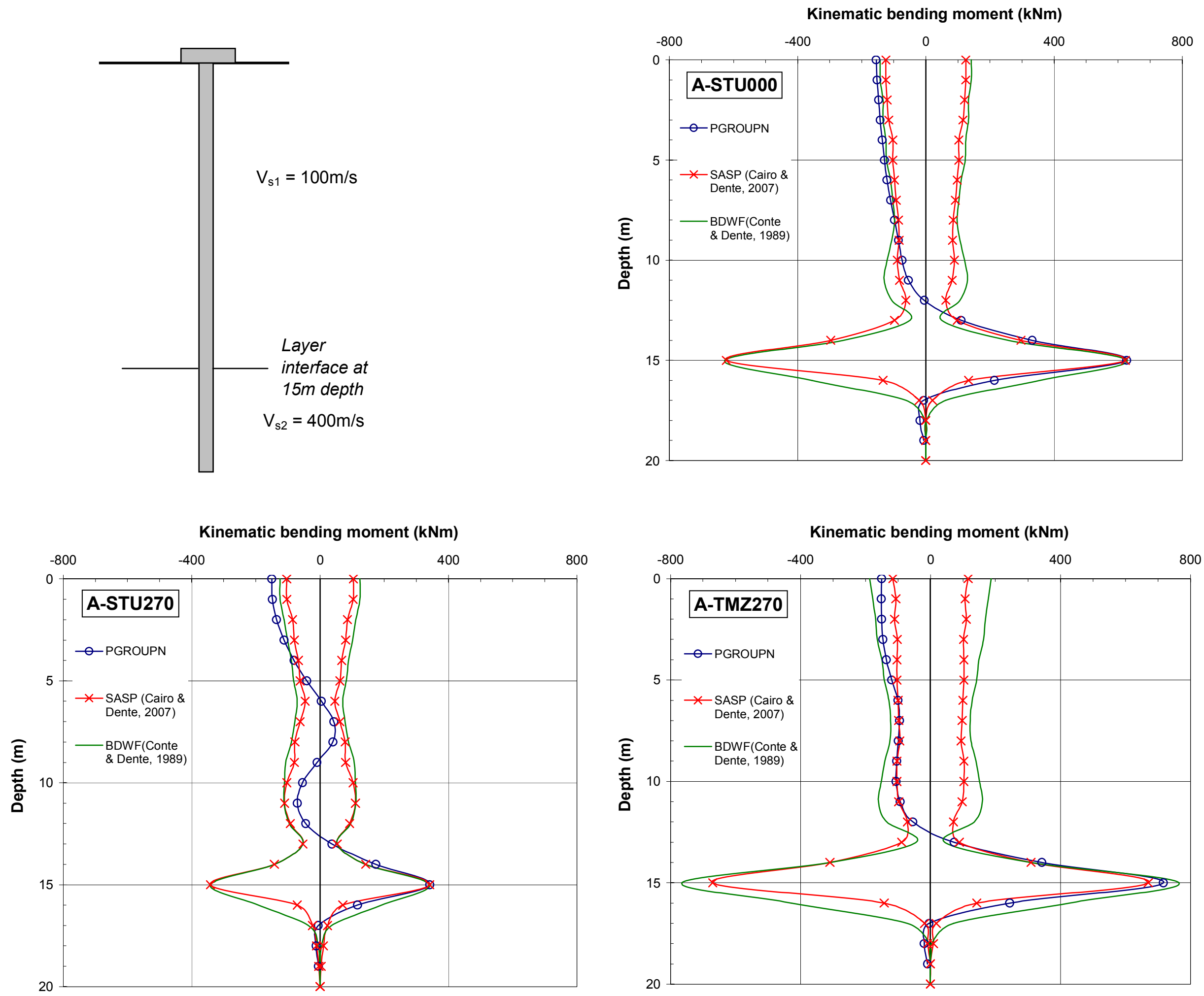

Fig. 11. Comparison of moment distribution with SASP and BDWF

The kinematic effects of pile-to-pile interaction as computed by PGROUPN are illustrated in Fig. 12 for a 3x3 pile group with a centre-to-centre spacing of three pile diameters. The pile and soil parameters are those reported in Fig. 9 (with the layer interface at 15 m depth), while the seismic motion is A-STU000. For comparison, the shear force and bending moment profiles computed for the single isolated pile have been included (no axial force is induced on the single pile due to horizontal ground movement). The following characteristics of behaviour can be discerned: (1) the corner piles of the group carry the greatest proportion of axial force, shear force and bending moment (similarly to the case of inertial loading), (2) the corner piles carry a smaller proportion of shear force (7%) and bending moment (10%) as compared to the single pile, (3) although no vertical load is applied to the group, axial forces develop in the piles to counteract rocking of the pile group.

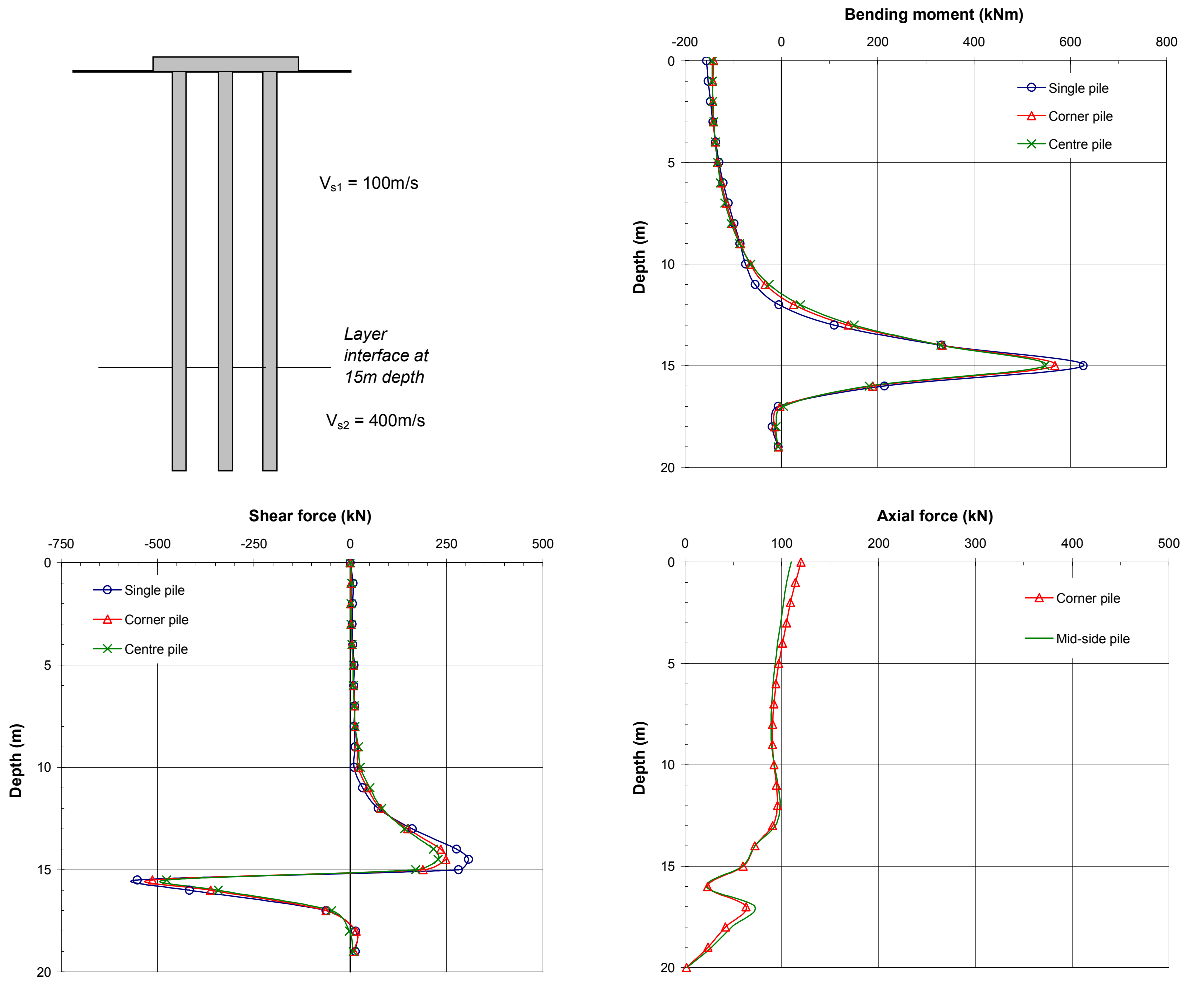

Fig. 12. PGROUPN prediction of internal forces in 3x3 pile group

Finally, Fig. 13 illustrates the effect of soil nonlinearity on the kinematic bending moment induced on the single isolated pile and on the corner pile of the 3x3 group. The results have been computed by PGROUPN under the assumption that the above two-layer soil profile is composed of clay material with an undrained shear strength (C_u) derived from the correlation E_s = 1000C_u and an adhesion factor (α) equal to 0.5 (the latter parameter is required to evaluate the non-linear rocking response of the group). The PGROUPN non-linear analysis has been preceded by a non-linear free-field EERA analysis using an initial damping ratio D_o equal to 0.5% and the default degradation curves for the shear modulus and damping ratio for clay (Seed & Sun, 1989; Idriss, 1990). Fig. 14 shows the maximum acceleration and lateral movement profiles obtained from the linear elastic (LE) and non-linear (NL) free-field EERA analyses. For comparison, the moment profiles from the linear elastic PGROUPN analyses (based on the linear elastic EERA analysis), as already reported in Fig. 12, are also included in Fig. 13. In addition, the results from a linear PGROUPN analysis based on a non-linear EERA analysis are shown. A number of features emerges from this figure: (1) kinematic pile-to-pile interaction leads to a reduction of the induced bending moment as compared to a single isolated pile, thereby confirming the trend observed in the case of linear elastic soil; (2) consideration of soil nonlinearity effects can have a significant influence on the kinematic bending moment of both single piles and pile groups; (3) the difference between the single-pile moment profile obtained from the non-linear PGROUPN analysis (preceded by a non-linear EERA analysis) and that obtained from the linear elastic PGROUPN analysis (preceded by an identical non-linear EERA analysis) shows that the overall nonlinearity of response is determined not only by the nonlinearities due to the shear waves propagating in the free-field soil but also by the nonlinearities due to pile-soil interaction.

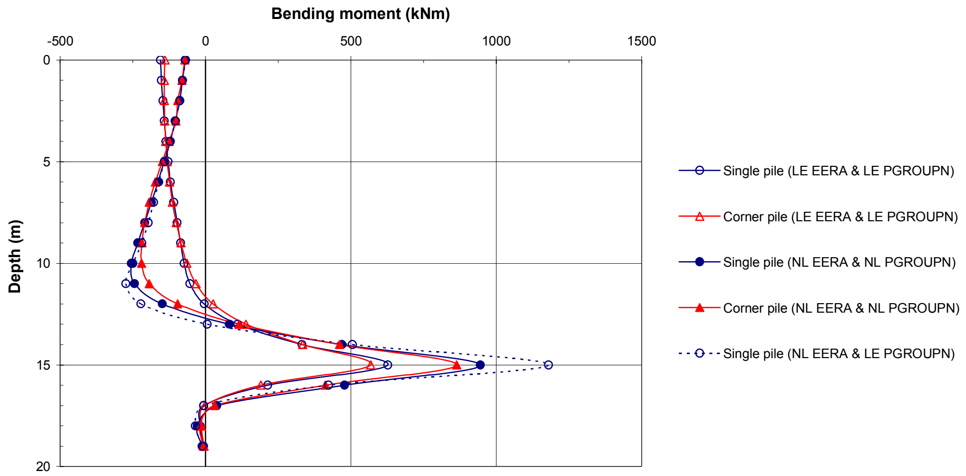

Fig. 13. Influence of soil nonlinearity on bending moment distribution

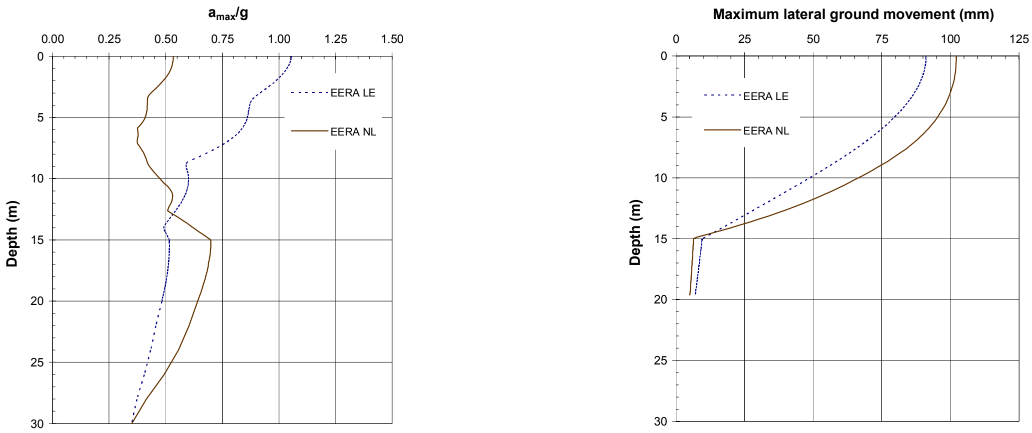

Fig. 14. Maximum free-field acceleration and lateral movement profiles

5. Conclusions

A relatively simple pseudostatic procedure for estimating the axial, lateral, and rocking behaviour of single piles and pile groups subjected to seismic excitation has been suggested. The approach involves a preliminary free-field seismic analysis to obtain the maximum ground displacements, and then, by means of a complete non-linear boundary element analysis (implemented in the code PGROUPN), the computed ground displacements are applied statically to the piles (kinematic loading) in addition to the static load at the pile head (inertial loading). Validity of the approach has been assessed by comparison with alternative numerical procedures. Based on the results presented in the paper, a number of considerations may be made:

● The proposed approach yields predictions of the pile internal forces which are consistent with those provided by other methods, ranging from simple Winkler models to sophisticated dynamic analyses. In addition, it involves negligible computational costs (both in terms of data preparation and computer execution times), allowing parametric studies to be readily performed. The results suggest that the method has promise in real design, offering a practical compromise between the uncertainty and limitations of Winkler models and the complexity and time-consuming nature of rigorous dynamic FEM or BEM analyses.

● Kinematic effects may have a significant influence on the single-pile internal forces, particularly when the subsoil profile includes layers with marked differences in stiffness. Such effects may be important not only for ground types D or worse, as recommended by Eurocode 8, but also for ground type C. Kinematic pile-to-pile interaction, generally ignored by current analysis methods, appears to affect the load distribution in a pile group, specifically by decreasing the induced shear forces and bending moments (i.e. a beneficial effect), and by increasing the induced axial forces (i.e. a detrimental effect) as compared to a single isolated pile.

● Simultaneous consideration of kinematic and inertial loading is recommended, particularly when significant soil nonlinearity is expected and hence the application of the superposition principle becomes less accurate.

● Available closed-form expressions, in addition to a number of practical limitations, generally yield a significant overprediction of kinematic bending moment at the soil layer interface.

● In addition to the nonlinearity arising from the passage of the seismic waves in the free-field soil, nonlinearity due to pile-soil interaction can have significant influence on both the inertial and kinematic distribution of pile internal forces and should not routinely be disregarded.

● Pile-soil interaction is a three-dimensional problem and each of the load components has deformation-coupling effects. Modelling of this aspect is crucial in real design (where the pile group is normally subjected to a simultaneous combination of axial forces, lateral forces and bending moments, in addition to lateral seismic excitation), thereby allowing a more realistic prediction of pile-group response.

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