Paper No. 11.01

SECOND INTERNATIONAL CONFERENCE ON PERFORMANCE-BASED DESIGN IN EARTHQUAKE GEOTECHNICAL ENGINEERING

May 28-30, 2012 - TAORMINA (ITALY)

PILE-GROUP RESPONSE UNDER SEISMIC LOADING

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ABSTRACT

Performance-based design requires the use of numerical analysis in order to effectively predict the pile response during earthquakes. In an attempt to provide a practical tool for the pile designer, this paper presents a pseudostatic approach for estimating the deformation behaviour and internal forces of pile groups under seismic excitation. The approach is capable of accounting for both inertial and kinematic effects by means of a two-step procedure: (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 cap based on the maximum surface acceleration (inertial loading). Validity of the approach is illustrated through comparison with numerical analyses and field measurements. The results indicate that the method has promise in practical applications, offering a reasonable compromise between the fundamental limitations of Winkler models and the complexity and time-consuming nature of dynamic analyses.

Keywords: Pile, pile group, seismic, pseudostatic, inertial, kinematic

INTRODUCTION

Seismic design of pile foundations is often based on pseudostatic approaches which only consider the pile-head inertial forces generated from the oscillation of the superstructure while neglecting the effects of kinematic interaction, i.e. the effects arising from the passage of seismic waves through the surrounding soil. However, post-earthquake field investigations have demonstrated the significant role of kinematic effects in the development of pile damage (Mizuno, 1987; Tazoh et al., 1987). Indeed, under certain conditions, the vibrating soil may impose large curvatures to the piles which in turn generate forces and, in particular, bending moments; such moments develop even in the absence of a superstructure and are referred to as "*kinematic*" moments, to be distinguished from moments generated by structural loading at the pile head ("*inertial*" moments). It is therefore essential to consider both inertial and kinematic effects in order to reliably predict the response of piled structures under seismic loading and meet the demands of performance-based design.

The importance of kinematic effects has recently been recognized by seismic regulations such as Eurocode 8 (EN 1998-5) which 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*".

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While there is ample geotechnical experience on carrying out pseudostatic analyses for the inertial loading, no specific method of analysis is readily available to the designer in order to evaluate the pile response to kinematic loading. Although a number of analysis methods has been presented, ranging from simplified approaches to sophisticated dynamic finite element formulations, there is a need to better understand the kinematic interaction effect and to develop reliable and efficient methods for predicting the pile behaviour in seismic conditions. A practical pseudostatic approach for estimating the deformation behaviour and internal forces of single piles and pile groups subjected to earthquake loading has recently been proposed by Basile (2010a, 2010b). The approach is capable of accounting for both inertial and kinematic effects by means of a two-step procedure: (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 cap based on the maximum surface acceleration (inertial loading). In this paper, validity of the approach is assessed through comparison with alternative numerical analyses and field measurements.

OVERVIEW OF ANALYSIS METHODS

The complexity of the problem of seismic pile-soil interaction requires the use of computer-based methods of analysis. 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; (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 imposes on the foundation. The above decomposition of the problem does not necessarily imply that the three steps must be performed separately, and a *complete* interaction 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 would be prohibitive for design, particularly when the effects of soil nonlinearity under seismic excitation are significant.

Numerical methods of analysis for estimating the deformation behaviour and internal forces of pile foundations under seismic excitation may be broadly classified into two categories: (1) Winkler-type approaches, and (2) continuum-based approaches, as described below.

Winkler model

This category 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 generally been derived through calibration against results of rigorous continuum-based dynamic analyses. While this approach has been used extensively to estimate the dynamic impedance of piles in relation to inertial interaction analyses, several studies have also employed the Winkler model to determine the kinematic response of piles (Kavvadas & Gazetas, 1993; Mylonakis et al., 1997; Nikolaou et al., 2001).

Based on the above methodology, some authors have proposed simplified closed-form expressions for estimating the kinematic pile bending moment at the interface between two soil layers (e.g. Dobry & O'Rourke, 1983; Nikolaou et al., 2001; Mylonakis, 2001). However, these simplified expressions are more suitable for the preliminary design stages due to a number of shortcomings, including 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, which is typically of the order of 10 to 15 pile diameters below the ground surface).

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

(a) *Single-pile response*
The Winkler model is of semi-empirical nature in that the spring coefficient "*k*" 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. Its value therefore depends not only on the soil properties but also on the pile properties and geometry. Thus, the evaluation of the spring constant for a specific pile and soil type has many uncertainties and a large amount of engineering judgement is needed (e.g. Poulos et al., 2001; Basile, 2003; Finn, 2005). It is worth noting 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 major 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 independent springs (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 often 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 several limitations (e.g. Basile, 2003); its use becomes even more questionable 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 always insignificant (Nikolaou et al., 2001; Dezi et al., 2009, Elahi et al., 2010).

(c) *Load-deformation coupling*
Pile-soil interaction is a truly 3D 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 combination of axial forces, lateral forces and moments), cannot readily be modelled by the Winkler approach.

Continuum-based approach
The fundamental limitations of the Winkler model may be removed by means of continuum-based dynamic solutions, generally based on the boundary element (BEM) or the finite element method (FEM) (e.g. Maheshwari et al., 2004). These analyses are capable of retaining the essential aspects of pile interaction through the soil continuum and hence a more realistic representation of the problem. Further, the mechanical characteristics to be introduced into the model have now a clear physical meaning (e.g. the soil Young's modulus E_s) and can be measured directly. However, dynamic solutions are very complex to use for design purposes and burdened by the large number of parameters and expertise needed for their execution, particularly when non-linear behaviour is to be considered. Major problems are related to the high mesh dependency and to the uncertain in assigning mechanical properties to the pile-soil interface elements. In

addition, the three-dimensional nature of the pile-group problem makes the analysis prohibitively expensive from a computational viewpoint for design purposes. This is particularly relevant for the dynamic case in which the disturbance travels as a wave in the ground and, contrary to the static case (where the load influence is confined to a limited area around the load application point), 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.

In order to overcome the shortcomings of dynamic analyses, BEM-based pseudostatic approaches have recently emerged (e.g. Tabesh & Poulos, 2001; Liyanapathirana & Poulos, 2005; Elahi et al., 2010). Unlike dynamic analyses which attempt to simulate the complete pile-soil interaction during seismic excitation in order to evaluate the "*time variation*" of pile response (i.e. deformations, forces and moments), pseudostatic approaches only aim at estimating the "*maximum*" pile response by using a relatively simple model and a small number of conventional engineering parameters. 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.

Pseudostatic approaches, to be effective, must be capable of accounting for both kinematic and inertial effects. This may be accomplished by using a two-step procedure: (1) computation of the maximum soil movements via a free-field seismic analysis and, (2) by means of a static boundary element analysis, computation of the pile response subjected to the simultaneous application of the computed free-field soil movements (kinematic loading), in addition to the static loading at the pile head (inertial loading). The rationale for the approach follows from the work of Tabesh & Poulos (2001) who 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. These findings are confirmed by the shaking table tests of Tokimatsu et al. (2005) which show that, if the natural period of the pile-cap-soil system is less than that of the soil deposit, kinematic and inertial loading tend to be in phase and act on the pile at the same time.

With the increase in cap-mass (and hence in its natural period), Tabesh & Poulos (2001) show that 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 are assumed to act simultaneously (i.e. in phase), which does not occur in a dynamic analysis. It is worth noting that the suitability of adopting pseudostatic approaches (i.e. zero frequency) in seismic design also stems from the consideration that single-pile stiffness is only slightly affected by loading frequency and therefore its static stiffness applies for practical use (Gazetas, 1991; Pecker & Pender, 2000). As for group interaction effects, although these are, in general, frequency dependent (Kaynia & Kausel, 1982; Gazetas et al., 1991), it has been shown that they do not fluctuate appreciably with frequency within the low-frequency range generally associated with earthquakes (Prakash et al., 1996). Similar conclusions were reached by Pecker & Pender (2000) and Zhang et al. (2004).

PGROUPN ANALYSIS

The pseudostatic approach recently proposed by Basile (2010a, 2010b) extends the outlined procedure of Tabesh & Poulos (2001), valid for single piles in linear elastic soil, in order to include the effects of group interaction and soil nonlinearity. The numerical procedure is carried out within PGROUPN (Basile, 2003), a computer program for determining the axial, lateral, rocking, and torsional response of pile groups based on a non-linear BEM solution of the soil continuum. The analysis involves discretization of only the pile-soil interface into a number of elements and adopts the traditional Mindlin solution 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. Non-linear response at the pile-soil interface is simulated by adopting a hyperbolic stress-strain model within a stepwise incremental procedure which ensures that the specified limiting stresses at the pile-soil interface are not exceeded. The program has negligible computational costs and is widely used in pile group design through the software Repute (Bond & Basile, 2010). The approach involves two main steps:

(1) Preliminary free-field site response analysis using available codes (such as SHAKE or EERA) in order to obtain both the maximum ground displacement profile (at the location of the pile element centres) and the maximum ground surface acceleration generated by the earthquake.

(2) PGROUPN static analysis of the pile group subjected to the simultaneous application of the computed maximum free-field ground displacement profile along the piles (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), as illustrated in Fig. 1. It is noted that the maximum values of the ground displacements along the piles are treated as a static ground movement profile, even though the maximum displacements at each element centre may have occurred at different times.

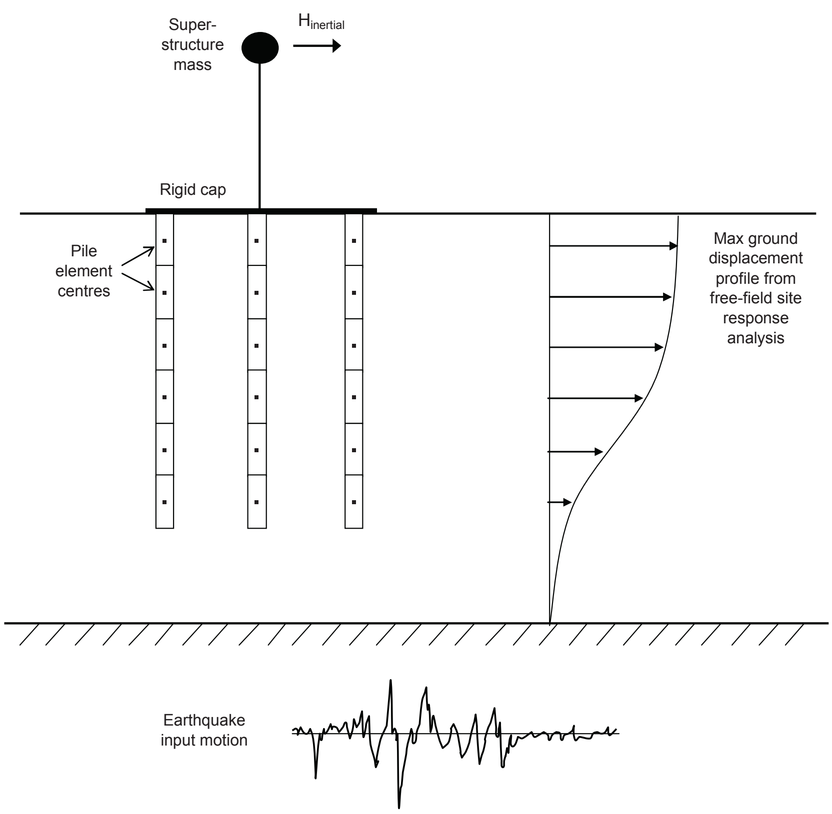


Figure 1. Pseudostatic BEM schematisation of the problem

NUMERICAL RESULTS

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

Comparison with Poulos (2006a)

The crucial difference between the effects of direct applied loading (i.e. inertial-type) and loading induced by external ground movements (i.e. kinematic-type) is examined through the analysis of a fixed-head single pile in a layered soil profile, as shown in Fig. 2 (ground type C according to Eurocode 8). The site is assumed to be subjected to the 1994 Northridge (U.S.) earthquake (Sepulveda station, SPV270 record from PEER database, PGA = 0.753g), scaled to a maximum bedrock acceleration of 0.2g. The loading conditions consist in a lateral "inertial" load of 0.2 MN at the pile head and in the maximum lateral ground movement profile along the pile derived from a free-field EERA analysis ("kinematic" loading). Figure 2 compares the bending moments computed from PGROUPN with those obtained by Poulos (2006a) by means of a similar BEM-based pseudostatic procedure (employing an elastic-perfectly plastic pile-soil interface model). The results show the basic 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.

Comparison with dynamic analyses

The PGROUPN pseudostatic approach is compared with the beam-on-dynamic-Winkler-foundation (BDWF) solution by Conte & Dente (1989) and the rigorous dynamic BEM analysis by Cairo & Dente (2007) using the SASP code. Under the assumption of linear elastic soil behaviour, the effects of *kinematic* loading are illustrated for a fixed-head single pile embedded in a two-layer soil profile underlain by rigid bedrock (Fig. 3). It is assumed that the site is subjected to the 1980 Irpinia (Italy) earthquake (Sturno station, A-STU000 record from SISMA database, PGA = 0.223g), scaled to a maximum bedrock acceleration of 0.35g.

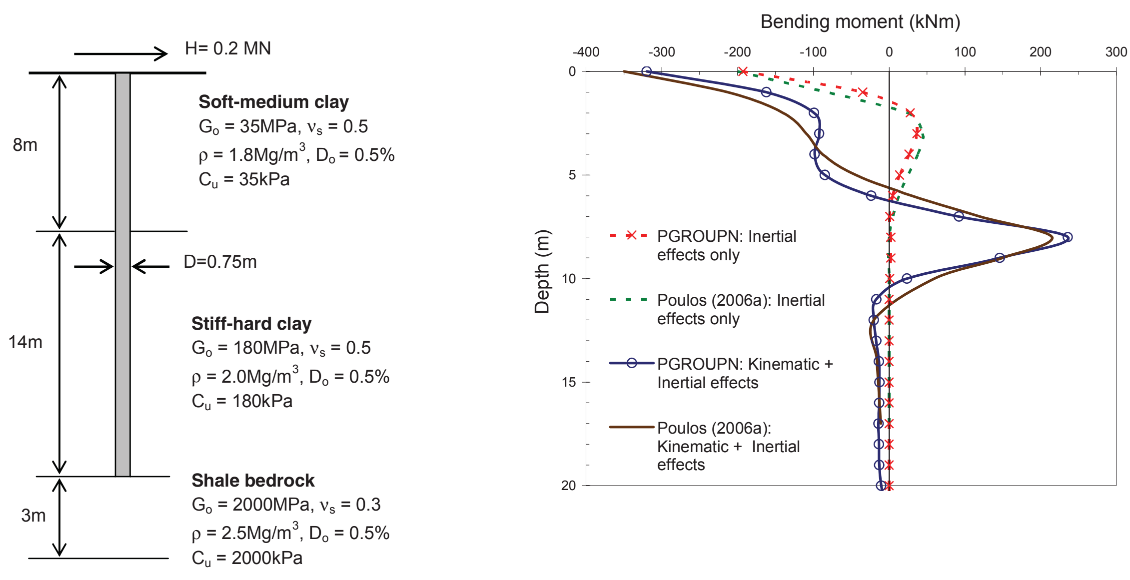


Figure 2. Comparison of bending moment with Poulos (2006a)

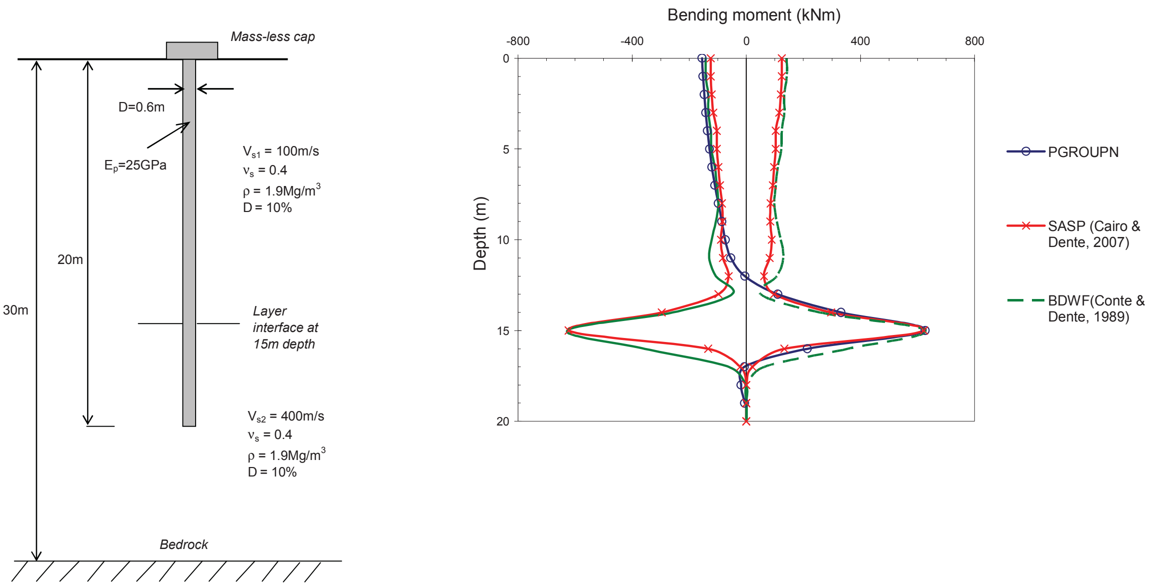


Figure 3. Comparison of kinematic bending moment with dynamic analyses

The kinematic bending moments are compared in Figure 3 in which, in order to facilitate the comparison with the pseudostatic PGROUPN analysis (which provides the maximum values along the pile), the envelope of the maximum positive moments calculated from the dynamic SASP and BDWF analyses 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).

The kinematic effects of pile-to-pile interaction as computed by PGROUPN are illustrated in Fig. 4 for a 3x3 pile group with a centre-to-centre spacing of three pile diameters, using the pile and soil parameters of Fig. 3. 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.

Figure 5 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 equal to 0.5 (the latter parameter is required to evaluate the non-linear rocking response of the group). The non-linear PGROUPN analysis has been preceded by a non-linear EERA analysis using an initial damping ratio D_o equal to 0.5%

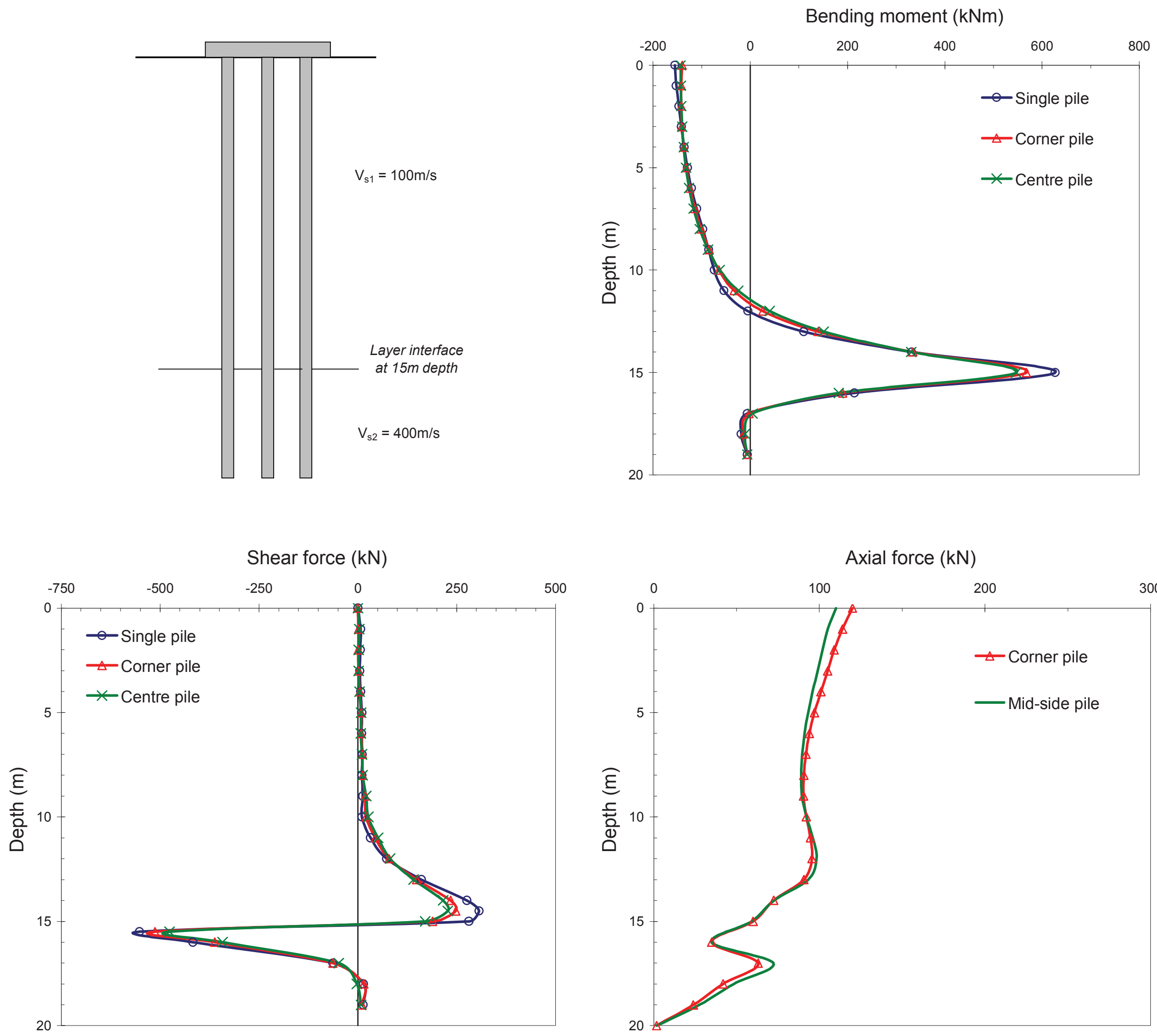


Figure 4. PGROUPN prediction of kinematic internal forces in 3x3 pile group

and the default degradation curves for the shear modulus and damping ratio for clay (Seed & Sun, 1989; Idriss, 1990). For comparison, the moment profiles from the linear elastic PGROUPN analyses (based on a linear elastic EERA analysis), as already reported in Fig. 4, are also included in Fig. 5. 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.

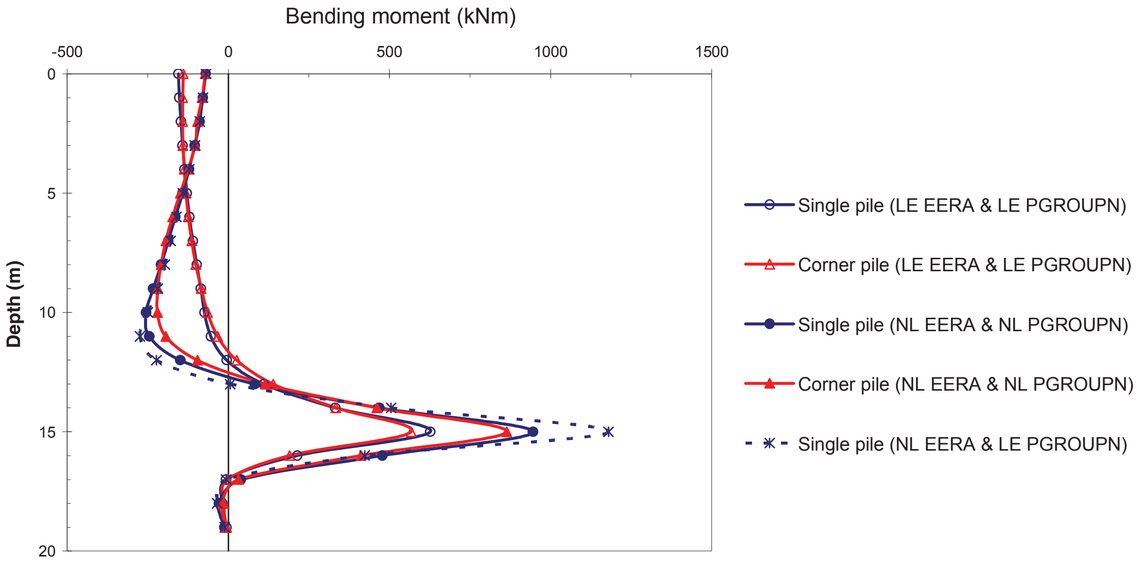


Figure 5. Influence of soil nonlinearity on kinematic bending moment

CASE STUDY: THE OHBA-OHASHI BRIDGE

The PGROUPN pseudostatic methodology is used to investigate the seismic response of a real instrumented pile-supported structure, specifically the Ohba-Ohashi Bridge located in Fujisawa City, Kanagawa prefecture, near Tokyo (Japan). The 485m-long, 16m wide bridge is supported on eleven piers founded on pile groups. Four piers (P5 to P8) are connected with a 143m-long continuous girder which is attached with a fixed-shoe connection to pier P6 and with movable bearings to the other piers. Fig. 6 shows the details of the pile foundations of pier P6 (where the strain meters were installed) and the soil profile obtained from a nearby borehole. The top soil layers are extremely soft with SPT-N values approaching zero and shear wave velocities in the range 40-100 m/s. The bearing substratum consists in a stiff clay with SPT-N values greater than 50 and a shear wave velocity of 400 m/s. Pier 6 is supported by a group of 64 piles (in a 8x8 configuration), 32 of which are vertical and 32 raked at an angle of 11.3^{o} along the longitudinal direction of the structure. Piles are spaced at 1.5 m (centre-to-centre) in both directions throughout the group, leading to a spacing-to-diameter ratio of 2.5. Piles are steel pipes 22m long with an outside diameter of 0.60 m and a wall thickness of 9 mm for vertical piles and 12 mm for raked piles. The piles are securely embedded for about 2 m in the underlying substratum. Comprehensive instrumentation of the bridge was undertaken by the Institute of Technology of Shimizu Corporation (Tazoh et al., 1987). A total of 11 units of accelerometers were installed, with one unit (GS1) at the ground surface, four units (GB1-GB4) at the bearing substratum, three units (BS1-BS3) at the pile cap, two units (BR1 and BR3) at the piers and one unit (BR2) at the girder. In addition, a total of 32 strain meters were installed on one vertical (Pile 20) and one raked (Pile 4) piles at four different elevations (SA1-SA4 and SB1-SB4) in order to measure axial and bending strains. Seismic observations were carried out by Shimizu Corporation between 1981 and 1985. During this period, a total of fourteen earthquakes were recorded. The earthquake which registered the maximum acceleration at the ground surface is the Kanagawa-Yamanashi-Kenzakai earthquake and occurred on 8 August 1983 (Earthquake No. 12, JMA magnitude 6, epicentral distance from the site 42 km and focal depth 20 km), with the maximum acceleration value being 0.11g (at GS1 in the H1 direction).

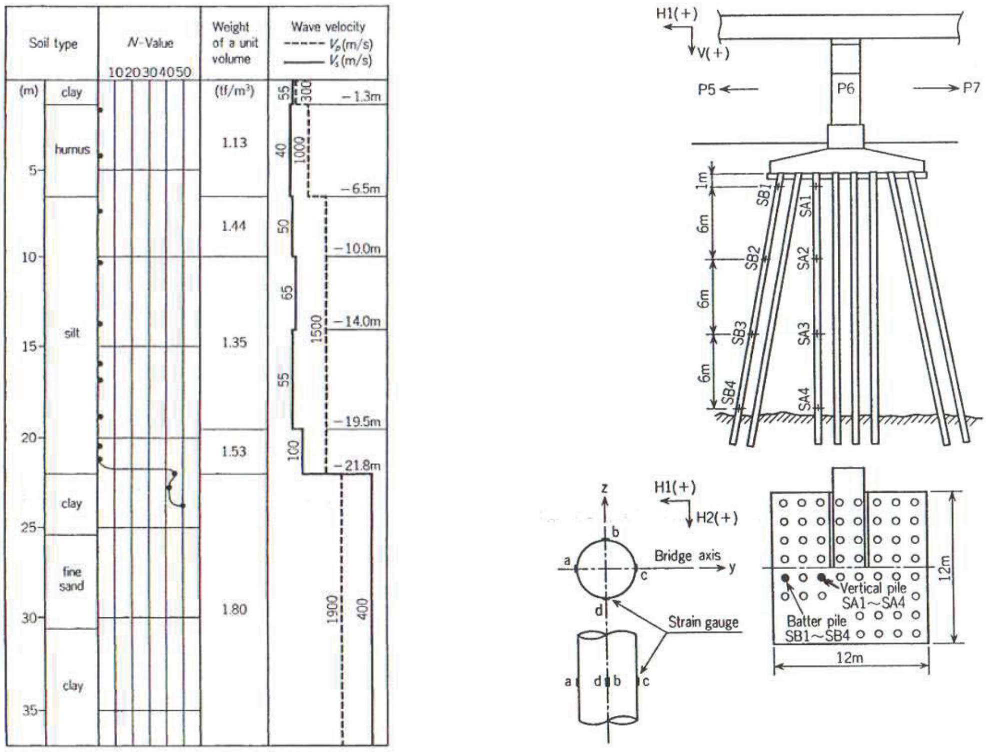


Figure 6. Ohba-Ohashi Bridge soil conditions and pile foundation (after Tazoh et al., 1987)

The PGROUPN pseudostatic procedure is used to predict the pile group response under Earthquake No. 12. For simplicity, also given the relatively low accelerations in the soil (max bedrock acceleration of about 0.03g at GB1 in the H1 direction), linear elastic soil behaviour is assumed. The soil is modelled as a seven-layer profile with Young's modulus E_s values derived from the properties reported in Fig. 6; in addition, a damping ratio of 3% and a Poisson's ratio of 0.5 have been assumed for all layers (Mylonakis & Syngros, 2005). In order to evaluate the kinematic-type loading, the maximum lateral ground movement profile along the vertical piles is computed by means of a free-field EERA analysis using the recorded motion at GB1 as the input motion, as shown on Fig. 7. For the raked piles, the maximum horizontal ground movement profile derived by EERA is resolved into its axial and lateral components. The inertial loading is estimated by concentrating the mass of the superstructure and the footing into two points, as reported by Tazoh et al. (1987). The mass m_1 of 2192 ton is defined as the sum of the weights of the continuous girder from piers P5 to P5 and half the weight of pier P6, while the mass m_0 of 801 ton is defined by adding half the weight of pier P6 to the weight of the footing. The heights (i.e. the eccentricities) of the above lumped mass above the pile-head level are estimated as 10m for m_1 and 1m for m_0. The maximum free-field accelerations at pile-head level computed from EERA are equal to 0.047g and 0.048g in the H1 and H2 directions, respectively. Thus, with these assumptions, the PGROUPN static analysis is performed for a pile-group subjected to the simultaneous application of a lateral inertial force in H1 direction of 1380 kN, an overturning moment (caused by eccentricity of the lateral force) of 10476 kNm, a lateral inertial force in H2 direction of 1409 kN, an overturning moment of 10699 kNm, and the maximum ground movement profiles along the piles computed by EERA.

Figure 8 reports the envelopes of maximum positive and negative bending moments in H1 (i.e. M_z) and H2 (i.e. M_y) directions at four locations along the vertical (No. 20) and the raked (No. 4) piles, as derived from

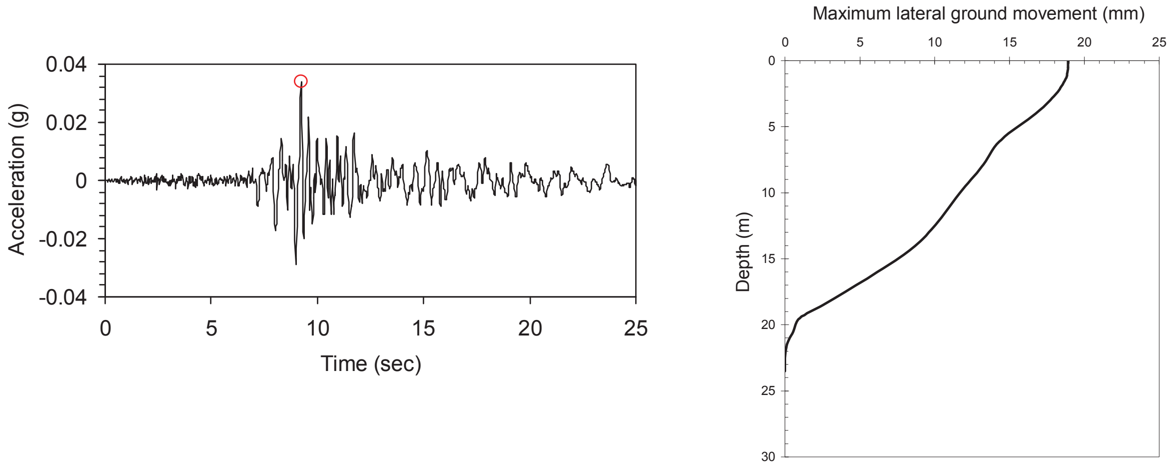


Figure 7. Recorded seismic motion at GB1 (H1 direction) and resulting maximum lateral ground movement profile from EERA

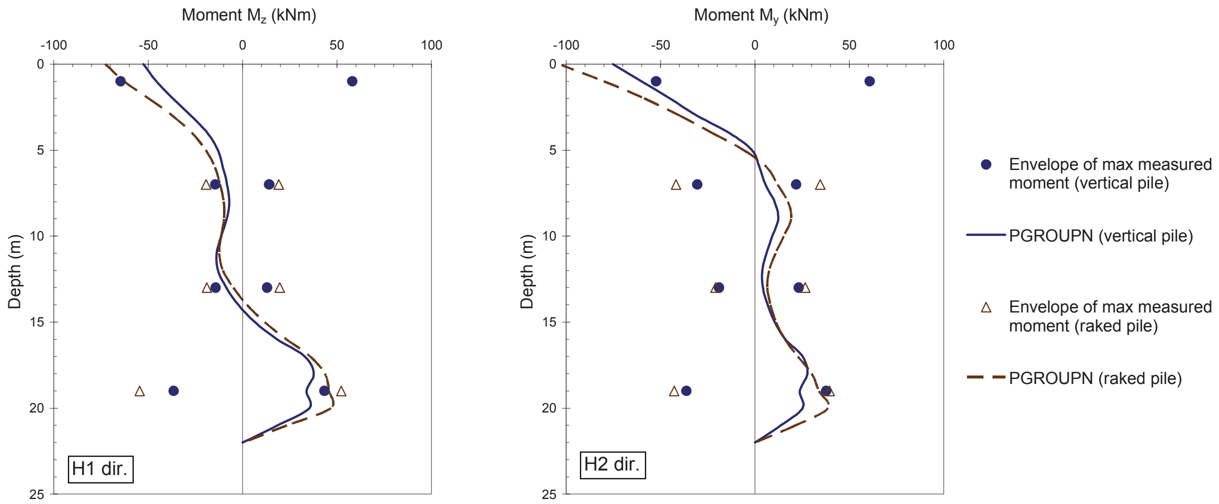
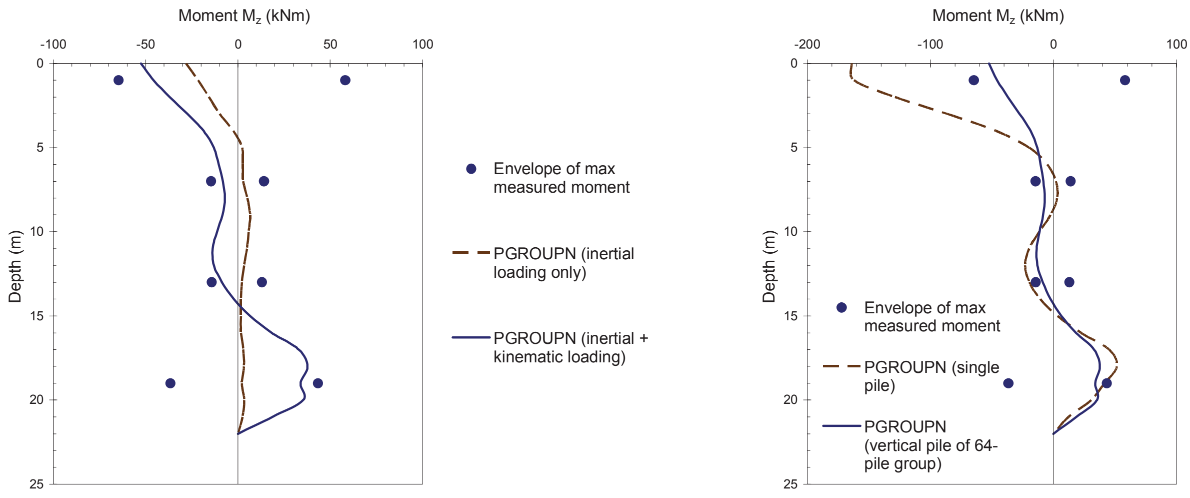


Figure 8. Comparison of bending moment in H1 and H2 directions

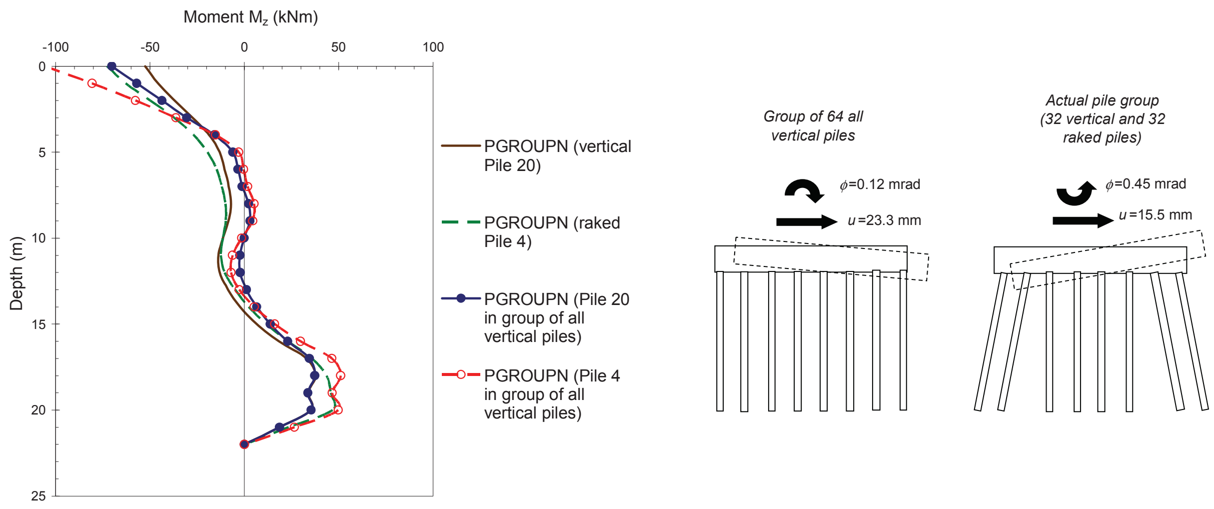
the recorded dimensionless bending strains ε_{pB} (using the correlation $\varepsilon_{pB} = Md/2E_pI_p$, where d is the pile diameter). For comparison, Fig. 8 shows the bending moment profiles computed by the pseudostatic PGROUPN analysis (which represent maximum values along the pile). A good agreement between measured and calculated moments is obtained, with the highest values occurring at the pile-head and at the proximity of the interface between the soft layer and the stiff bearing substratum, as expected.

Figure 9a compares the bending moment profile in H1 direction for the vertical pile (Pile 20), including the profile obtained by a PGROUPN analysis ignoring the effects of kinematic loading. It is noted that kinematic and inertial loading contribute in approximately equal proportions to the pile-head moment, whereas the moment near the layer interface is solely produced by kinematic loading. This confirms the basic importance of accounting for both inertial and kinematic effects in the evaluation of pile bending moment. The effects of pile-to-pile interaction are highlighted in Figure 9b which includes the moment profile from a simple single-pile analysis in which the pile is subjected to the inertial loads taken as equal to 1/64 of the above group



(*a*) (*b*)

Figure 9. (*a*) Effect of kinematic and inertial loading; (*b*) Effect of group interaction



(*a*) (*b*)

Figure 10. Effect of raked piles on (*a*) bending moment and (*b*) cap deflection and rotation

loads (i.e. under the assumption that all piles in the group carry equal loads), in addition to the EERA maximum lateral ground movement profile. It is noted that, due to the effects of group interaction, the single-pile analysis overestimates the bending moments at the pile-head (in particular) and near the layer interface, thereby showing the shortcomings of attempting to model a pile group as an equivalent single pile.

The effect of rake on the pile response in H1 direction is analysed using PGROUPN. Figure 8 had shown that both measurements and analysis give larger bending moments in the raked pile (Pile 4) than in the vertical pile (Pile 20). In order to investigate this aspect further, Figure 10a also includes the bending moment profiles induced on the corresponding piles (i.e. Piles 4 and 20) of a group of 64 all vertical piles. If attention is focused on Pile 4, it is noted that raking the pile actually decreases the head bending moment (i.e. a beneficial effect), while the kinematic bending moment at the layer interface remains substantially unaltered. This shows that the larger bending moment induced on the raked pile (Pile 4) in the real pile-group configuration (i.e. 32 vertical and 32 raked piles) is actually caused by its location within the group (i.e. by group interaction effects) rather than by its rake. The influence of raked piles on the cap deflection and rotation is schematically illustrated in Fig. 10b which shows the snapshots of the actual (i.e. 32 vertical and 32 raked piles) and all vertical pile-group configurations. It is noted that the actual configuration with raked piles undergoes a smaller deflection than the vertical group, but at the same time it develops a cap rotation which is about four times greater than and in opposite direction to that of the vertical group. This feature of behaviour has been confirmed by several studies, including the static BEM analysis of Poulos (2006b), the dynamic FEM analysis of Giannakou et al. (2010) and the centrifuge tests of Tazoh et al. (2011).

The axial force induced on the piles by the earthquake loading to counteract rocking of the pile group is reported in Figure 11. The figure compares PGROUPN results (due to both kinematic and inertial effects) with the envelopes of maximum measured axial force as derived from the recorded dimensionless axial strains ε_{pA} (using the correlation $\varepsilon_{pA} = P/E_pA_p$, where P is the axial force). Results from a PGROUPN analysis which only includes inertial loading are also reported. A good agreement between theory and measurements is obtained, with the vertical pile attracting larger axial force than the raked pile. It is worth noting that PGROUPN is able to capture the increase in axial force with depth along the raked pile (which is shown to be caused by kinematic effects) and also the opposite sign of axial force in the vertical and the raked piles, as experimentally observed by Tazoh et al. (1987) during the entire duration of the earthquake.

The overall results of the comparison show that the PGROUPN pseudostatic analysis provides a reasonable estimate of the measured pile response at Ohba-Ohashi Bridge. This is despite the fact that both the soil conditions and the bridge structure are complex and have been highly simplified in the analysis. Indeed, the recorded motion used as input to the EERA analysis (i.e. at GB1) is some 70 metres offset with respect to the pile foundation even though it has been reported that the site is significantly affected by the local topography. In addition, previous studies demonstrated the significance of 3D valley effects for the site causing multiple reflections and refractions of the waves. Such effects cannot be reproduced by a one-dimensional site response analysis such as EERA. As for the superstructure modelling, its reduction to a concentrated mass is an approximation given that the structure is a multi-degree of freedom system with a far more complicated response pattern than a concentrated mass. As an example, the mere eccentricity of the superstructure mass may have a considerable effect on the loading conditions and hence on the pile response. As a final remark, it is noted that the PGROUPN pile-group analysis runs in 18 min using an ordinary computer (Intel Core i7 at 2.7 GHz with 6 GB RAM), which is an acceptable computing time for design purposes.

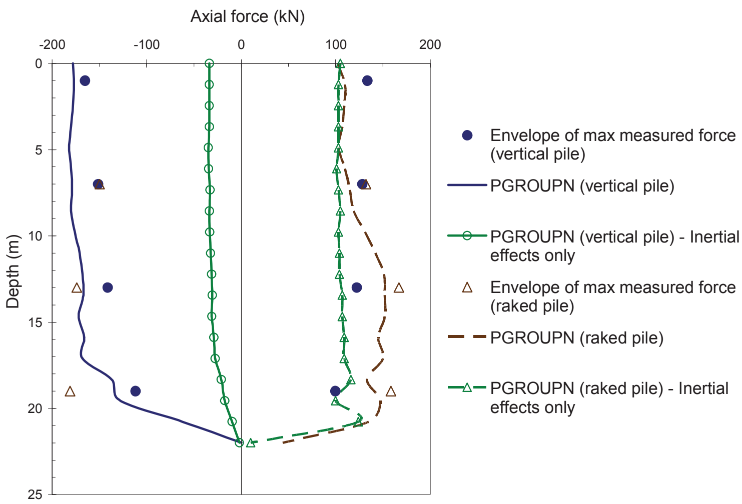


Figure 11. Comparison of axial force

CONCLUSIONS

In order to efficiently deliver the expected performance of a piled structure during earthquakes, it is essential to be able to predict the pile response in a reliable yet cost-effective fashion. In an attempt to provide a practical tool for the pile designer, this paper has presented a relatively simple pseudostatic approach for estimating the deformation behaviour and internal forces of a pile group subjected to seismic excitation. The approach is based on a two-step procedure: (1) computation of the maximum soil movements via a free-field seismic analysis and, (2) by means of a static boundary element analysis, computation of the pile response subjected to the simultaneous application of the computed free-field soil movements (kinematic loading), in addition to the static loading at the pile head (inertial loading). 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 in general agreement with those provided by alternative numerical analyses and field measurements. The approach involves relatively small computational costs (both in terms of data preparation and computer execution times), allowing parametric studies to be readily performed. The results suggest that the pseudostatic procedure has promise in practical applications, offering a reasonable compromise between the fundamental limitations of Winkler models and the complexity and time-consuming nature of rigorous dynamic analyses.

● Kinematic effects can 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 group interaction effects, generally ignored by current analysis methods, can 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.

● 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 a 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, torsional and bending moments, in addition to seismic excitation), thereby allowing a more realistic prediction of pile-group response.

ACKNOWLEDGEMENTS

The author is grateful to Dr. Takashi Tazoh, Shimizu Corporation, for his useful suggestions and for kindly supplying the seismic data on the Ohba-Ohashi Bridge. The help of Prof. George Gazetas, National Technical University of Athens, and Dr. Sissy Nikolaou, Mueser Rutledge Consulting Engineers, is gratefully acknowledged.

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