Bull Earthquake Eng (2008) 6:705–722
DOI 10.1007/s10518-008-9085-2

ORIGINAL RESEARCH PAPER

Evidence of beneficial role of inclined piles: observations and summary of numerical analyses

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Received: 22 October 2007 / Accepted: 19 August 2008 / Published online: 16 September 2008
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Abstract The behaviour under seismic loading of inclined piles embedded in two idealized soil profiles, a homogeneous and a non-homogenous "Gibson" soil, is analysed with 3D finite elements. Two structures, modeled as single-degree-of-freedom oscillators, are studied: (1) a tall slender superstructure ($H_{st} = 12$ m) whose crucial loading is the overturning moment, and (2) a short structure ($H_{st} = 1$ m) whose crucial loading is the shear force. Three simple two-pile group are studied: (a) one comprising a vertical pile and a pile inclined at 25°, (b) one consisting of two piles symmetrically inclined at 25°, and (c) a group of two vertical piles. The influence of key parameters is analysed and non-dimensional diagrams are presented to illustrate the role of raked piles on pile and structure response. It is shown that this role can be beneficial or detrimental depending on a number of factors, including the slenderness of the superstructure and the type of pile-to-cap connection.

Keywords Inclined pile · 3D finite element model · Seismic response · Group of piles · Pile-to-cap connection · Soil–pile–bridge pier interaction · Field observations

1 Introduction

Inclined piles find frequent use in foundations when substantial lateral stiffness is required. However for years, the seismic behaviour of inclined piles has been considered detrimental, and many codes require that such piles be avoided [French Seismic Code AFPS 1990; Eurocode EC8 2003/Part 5/Section 5.4.2(5)]. The main arguments that have been frequently mentioned by engineers as the (perceived) drawbacks of inclined piles include:

- "parasitic" bending stresses due to soil settlement (following an earthquake) and/or; soil consolidation (before the earthquake)

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- large forces (of alternating sign) onto the pile cap
- reduction in bending capacity due to seismically induced tensile forces
- undesirable permanent rotation of the cap when the inclination of the piles is not symmetric.

Related studies have been presented by Sadek and Shahrour (2004, 2006), Okawa et al. (2005), Poulos (2006), Deng et al. (2007), and Ravazi et al. (2007). Poulos (2006) studied the response of a 3 × 2 pile group subjected to lateral ground movement and found out that an increase in pile batter leads to a reduction in settlement, to a significant increase in pile–cap rotation, and an increase in axial force and bending moment at the pile head. Sadek and Shahrour (2006) studied the seismic response of inclined micropiles and showed that the symmetrically inclined micropiles of a 4 × 4 group supporting a short structure develop smaller bending moments and larger axial forces compared to a similar group of vertical micropiles. Deng et al. (2007) performed analyses for a large pile group containing inclined piles and showed that kinematic loading can have a major impact on the magnitude of the maximum axial force that developed in the batter piles. In their study, inclined piles developed 5–8 times greater axial forces than the vertical piles. Another problem that was found to be associated with the use of batter piles was the concentration of high axial forces leading in major damages in pile cap and/or pile head.

However evidence has been accumulating that inclined piles may, at least in certain cases and if properly designed, be beneficial rather than detrimental both for the structure they support and the piles themselves (Gazetas and Mylonakis 1998). Recent research (Guin 1997) has shown that the response of a typical bridge type structure to representative seismic excitation may improve in many respects when supported by inclined piles. Lam and Martin (1986) showed that both cap displacements and pile bending moments may be reduced dramatically in liquefied soil due to the stiffening effect of the inclined piles. In addition, field evidence has been accumulating revealing the successful performance of inclined piles. Characteristic examples include the Maya Warf (Kobe earthquake 1995), and the Landing Road bridge (New Zealand) in the Edgecumbe earthquake that are presented below (Berrill et al. 2001; Pender 1993). Until today the beneficial or detrimental role of batter piles to the seismic response of the superstructure or the foundation itself has not yet been clarified.

While most of the inclined piles suffered severe damage at the pile-to-cap connection, in several cases they may have contributed significantly to the successful performance of the supported bridges. These observations provided the motivation to investigate numerically (with 3D finite element models) the seismic performance of groups with inclined piles. Three simple configurations are studied in this paper: (a) the first comprises one vertical pile and one inclined at 25°, (b) the second consists of two piles inclined at 25°, and (c) the third consists of two vertical piles. The latter is used as a reference for comparison with the batter pile groups for detecting the beneficial or detrimental role of pile inclination in the seismic behaviour of the foundation. Since the type of pile-to-cap connection has been found to greatly influence the performance of the foundation, analyses were conducted for both hinged and fixed-head conditions.

2 Observed seismic performance of inclined piles

2.1 Unsatisfactory performance

The 1989 Loma Prieta Earthquake ($M_w = 6.9$) yielded important observations on the seismic performance of piles (SEAOC 1991). The 16 in-square prestressed concrete batter piles

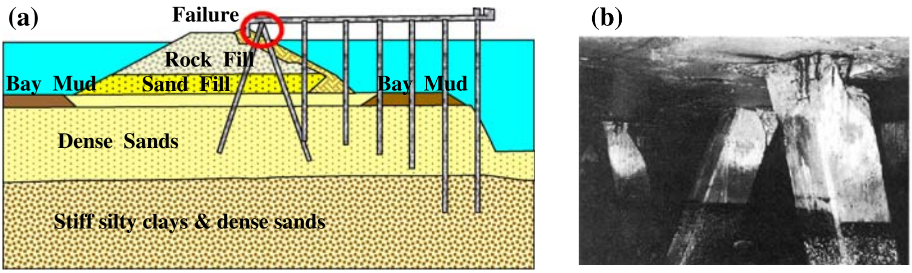


Fig. 1 7th Street terminal, Oakland, after the Loma Prieta earthquake: (**a**) cross section, (**b**) damage to batter piles (photo after SEAOC 1991)

supporting the Public Container Wharf at the 7th Street Terminal in the Port of Oakland failed in tension at their connection to the deck when the fill behind the wharf liquefied (Fig. 1). On the other hand, the vertical piles were largely undamaged with only a few exceptions. A similar type of failure was observed in the batter piles supporting the concrete wharf at the nearby Matson Terminal with additional damage to the back row of vertical piles. At the Oakland Outer Harbor Pier 7, 16-inch square prestressed concrete batter piles failed at or near the connection to the pile cap. In San Francisco, the Ferry Plaza Pier experienced tensile failure at the connection of the deck to the prestressed concrete batter piles, with some of the piles punching the slab. At Piers 27 and 29, similar damage occurred to over 120 of the 20 in square prestressed concrete batter piles.

These failures, however, were most probably due to inadequate reinforcement in the top of the piles and also improper connection of piles to their caps—a result of the early "isostatic" method of analysis which assumed that the piles transmit only axial load (Mitchell et al. 1991).

The magnitude 7.5 Costa Rica Earthquake caused severe damage over a large area, including liquefaction-related collapse of several pile-supported bridges. The front batter piles of the Rio Banano Bridge, suffered flexural and shear damage, whereas the vertical piles at the rear showed less damage—a consequence of the aforementioned misconception regarding the action of piles (Priestley et al. 1991). Similar type of damage of the inclined piles supporting the abutment of the Rio Vizcaya Bridge was observed leading to large rotations and collapse of the deck. Again, the mode of failure of the batter piles suggests that damage resulted from the insufficient design of the pile-to-cap connection (Priestley et al. 1991). Indeed, the case histories presented below, demonstrate that the role of inclined piles can often be beneficial rather than detrimental for the structure they support.

2.2 Satisfactory performance

One of the very few quay-walls that survived the disastrous 1995 ($M = 6.9$) Kobe earthquake in the harbour of Kobe was a composite wall, in Maya Wharf, supported by inclined piles. The presence of the inclined piles was one of the reasons why the quay wall managed to withstand the severe seismic motion and experience a displacement of only 20 cm (Kastranta et al. 1998). A nearby wall supported exclusively on vertical piles was completely devastated (displacements of about 3 m).

Berrill et al. (1997) conducted post earthquake investigations after the 1987 Edgecumbe Earthquake in New Zealand, and found out that a pier of the Landing Road Bridge withstood the large lateral ground movement (approximately 2 m) that had resulted from the liquefaction flow of a sandy layer, mainly due to the presence of inclined piles which offered the necessary lateral stiffness to the pile groups supporting the bridge.

3 Numerical analyses of the seismic response of inclined piles

3.1 Problem definition and model description

The problem under consideration is that of a structure supported alternatively by each of the three simple pile groups. Piles are characterized by Young's modulus E_p, diameter d, length L, and cross-sectional moment of inertia I_p. The center-to-center distance, s, between the piles at the pile-head level is 3 m. Two idealized soil deposits (Fig. 2) are analysed in this study: (a) a homogenous soil deposit whose Young's modulus is constant with depth (Profile I), and (b) a non-homogenous soil deposit whose Young's modulus is proportional to depth $[E_s(z) = E_s(d)z/d]$ (Profile II).

The problem is analysed in 3D, making use of the advanced Finite Element code ABAQUS. Both pile and soil are taken a linear material. The mass-and-column superstructure is modelled as a single degree of freedom oscillator. Two column heights are considered: 12 and 1 m. In both cases the concentrated mass, m_{st}, is 200 Mg, and the fixed base fundamental period of the structure T_{st} is 0.44 s. The fixed base fundamental period of the structure is set deliberately larger than the first natural period, $T_s = 0.3$ s, of both soil profiles used in the analysis. In this way spurious oscillations at the boundaries of the model are limited as a result of a *destructive interference* (existence of a cut-off period for radiation damping equal to the first natural period of the soil profile) of the outward spreading waves (Gerolymos and Gazetas 2006).

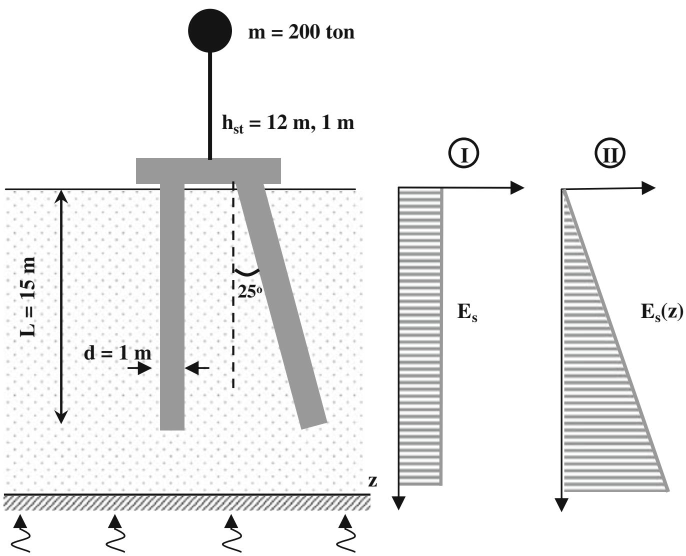


Fig. 2 Problem geometry and soil profiles

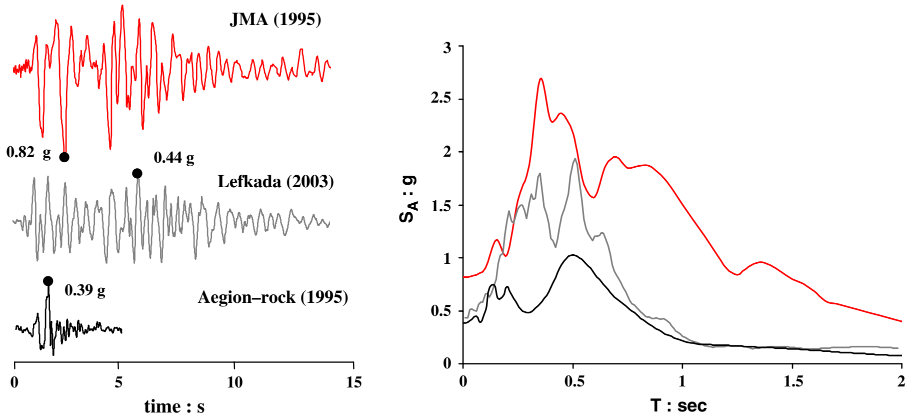


Fig. 3 Acceleration time histories used as base excitation and corresponding response spectra

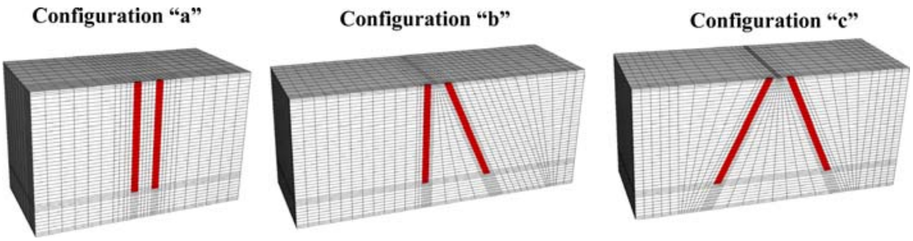


Fig. 4 Finite element discretisation of the three pile group configurations analysed

Soil and pile are modelled with eight-noded brick elements, while the superstructure is modelled with 3D beam elements. Thanks to symmetry, only one half of the model is analysed, thus significantly reducing computational demands. Figure 4 depicts the finite element discretization for the three studied configurations. Two pile-to-cap connections are considered, namely fixed-head and hinged-head piles. Rayleigh damping represents material damping, taken equal to 5% between the eigenfrequency of the soil deposit and the dominant frequency of the earthquake ground motion. Appropriate kinematic constraints are imposed to the lateral edges of the model, allowing it to move as the free field. Validation of the utilized finite element method for dynamic pile response analysis is provided in Giannakou et al. (2006, 2007).

Three real acceleration time histories covering a wide range of frequencies were used as base excitation (Fig. 3): (1) the Lefkada record, PGA $= 0.42$ g, from the $M = 6.4$ Lefkada 2003 earthquake, (2) the Aegion-rock outcrop motion, PGA $= 0.39$ g from the $M = 6.2$ Aegion 1995 earthquake, and (3) the JMA record, PGA $= 0.83$ g, from the $M = 7.2$ JMA Kobe 1995 earthquake. The dominant periods of the acceleration time histories at the free field surface for the aforementioned three earthquake records range from 0.3 to 0.8 s, resulting in a fixed base fundamental period ratio (designated as the fixed base fundamental period of the superstructure divided by the predominant period of the free field surface acceleration time history) which ranges from 0.55 to 1.46. This is a quite broad range of values which ensures generalization of the results presented herein.

Fig. 5 **a** Snapshots of contours of the horizontal displacement, plotted on the deformed mesh (×200), of the three pile group configurations for the non-homogenous soil profile (II) for fixed pile-to-cap connection. The models are subjected only to kinematic loading with Lefkada record as base excitation. **b** Snapshots of contours of the horizontal displacement, plotted on the deformed mesh (×200), of the three pile group configurations for the non-homogenous soil profile (II) for hinged pile-to-cap connection. The models are subjected only to kinematic loading with Lefkada record as base excitation

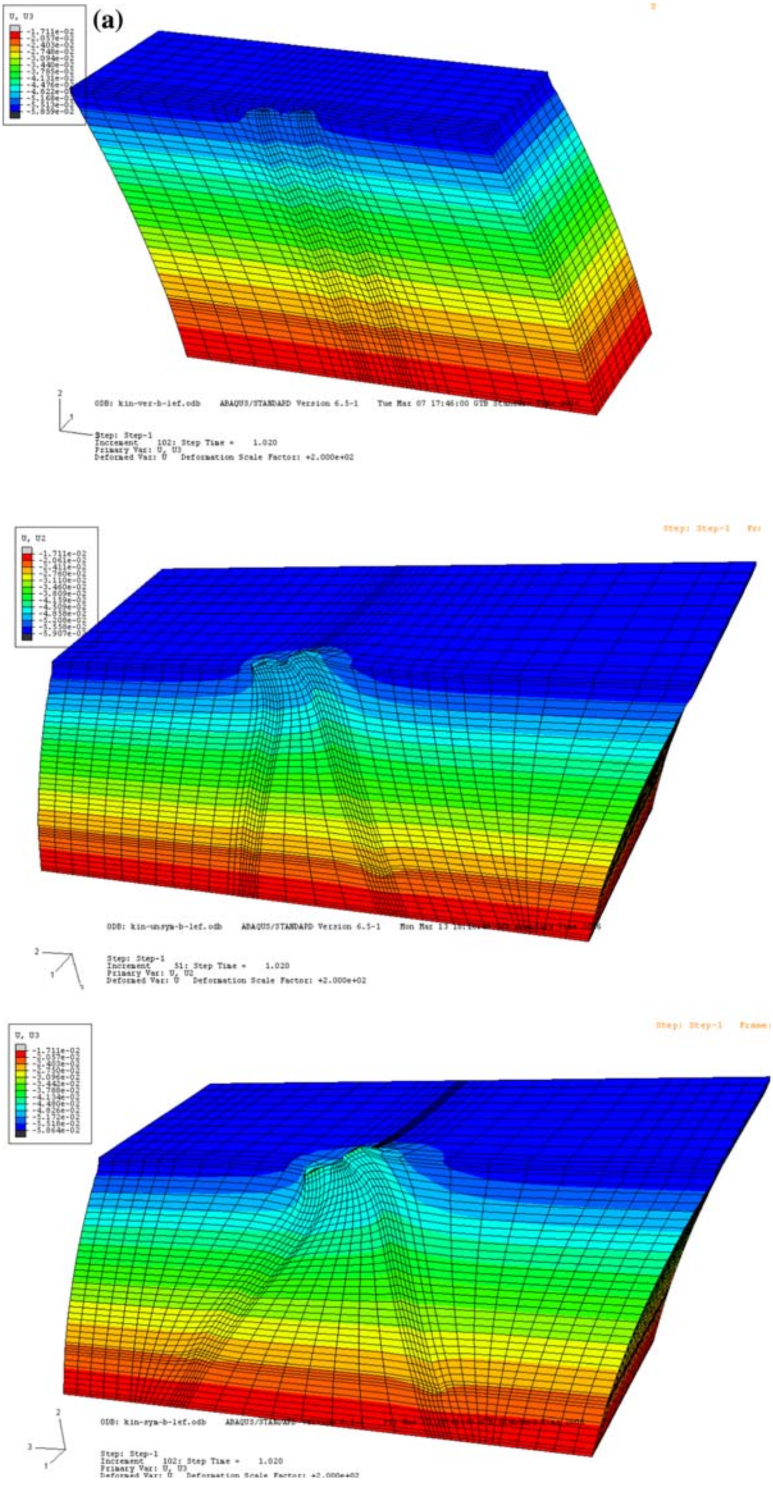
It is stated, however, that the role of key problem parameters on the system response, for example: pile-to-pile spacing, number of piles and pile configuration, inclination angle, soil stratigraphy, fundamental period of the superstructure, and pile and soil nonlinearities, is admittedly of considerable importance. Unfortunately, such a thorough parametric investigation is beyond the limits of a single paper as well as the scope of the present study.

4 Results and discussion

4.1 Kinematic response

Two sets of analysis were conducted: one for the kinematic response of the group, without the presence of a superstructure, and one for the inertial response of each structure supported alternately on each of the studied pile groups. Results of the kinematic response, for all earthquake records utilized, are presented in Fig. 5a, b in the form of selected snapshots of the deformed meshes and in Fig. 6 in the form of lateral displacements (U) and internal forces (M, N) normalized by their counterparts of the fixed-head vertical pile group. The results of the analyses lead to interesting (tentative) conclusions, the most important of which are summarized as follows:

- Using batter piles in a group increases the lateral stiffness and thereby reduces displacements during an earthquake (Fig. 6a). This is desirable when strict serviceability limits must be met. For fixed pile-to-cap connection, in both profiles, the displacements of the group with the two inclined piles are 40% less than those of the group with vertical piles. Such behavior matches well the frequently observed behaviour of inclined piles in the field under conditions of extensive soil movements (such as liquefaction and lateral spreading). The hinged connection, however, limits the aforementioned reduction leading to larger displacements of the pile groups.
- Pile-to-cap connection influences the performance of the foundation significantly, with the *hinged* connection leading to smaller pile "distress" at the cost of larger cap displacement and rotation than the fixed connection, as is illustrated in Fig. 5a.
- The bending moment that develops in the cap itself in the presence of inclined fixed-head piles is larger than in the vertical group. As illustrated in Fig. 6b, the hinged connection allows for a significant reduction in the bending moment in all configurations.
- The symmetrically inclined piles develop greater axial force which could be significantly larger than that of the group with vertical piles when the piles are embedded in the homogenous profile (Fig. 6c). The difference is attributed to the fact that profile II becomes significantly stiffer than profile I at greater depths and near the pile tip. Moreover, whereas the hinged connection leads to a reduction in axial force in the group of vertical piles, it does not seem to influence significantly the groups with batter piles.



(a)
U, U3
ODB: kin-ver-b-lef.odb ABAQUS/STANDARD Version 6.5-1 Tue Mar 07 17:46:00 GTB
Step: Step-1
Increment 102: Step Time = 1.020
Primary Var: U, U3
Deformed Var: U Deformation Scale Factor: +2.000e+02
U, U2
ODB: kin-unsym-b-lef.odb ABAQUS/STANDARD Version 6.5-1
Step: Step-1
Increment 51: Step Time = 1.020
Primary Var: U, U2
Deformed Var: U Deformation Scale Factor: +2.000e+02
U, U3
ODB: kin-sym-b-lef.odb
Step: Step-1
Increment 102: Step Time = 1.020
Primary Var: U, U3

Fig. 5a

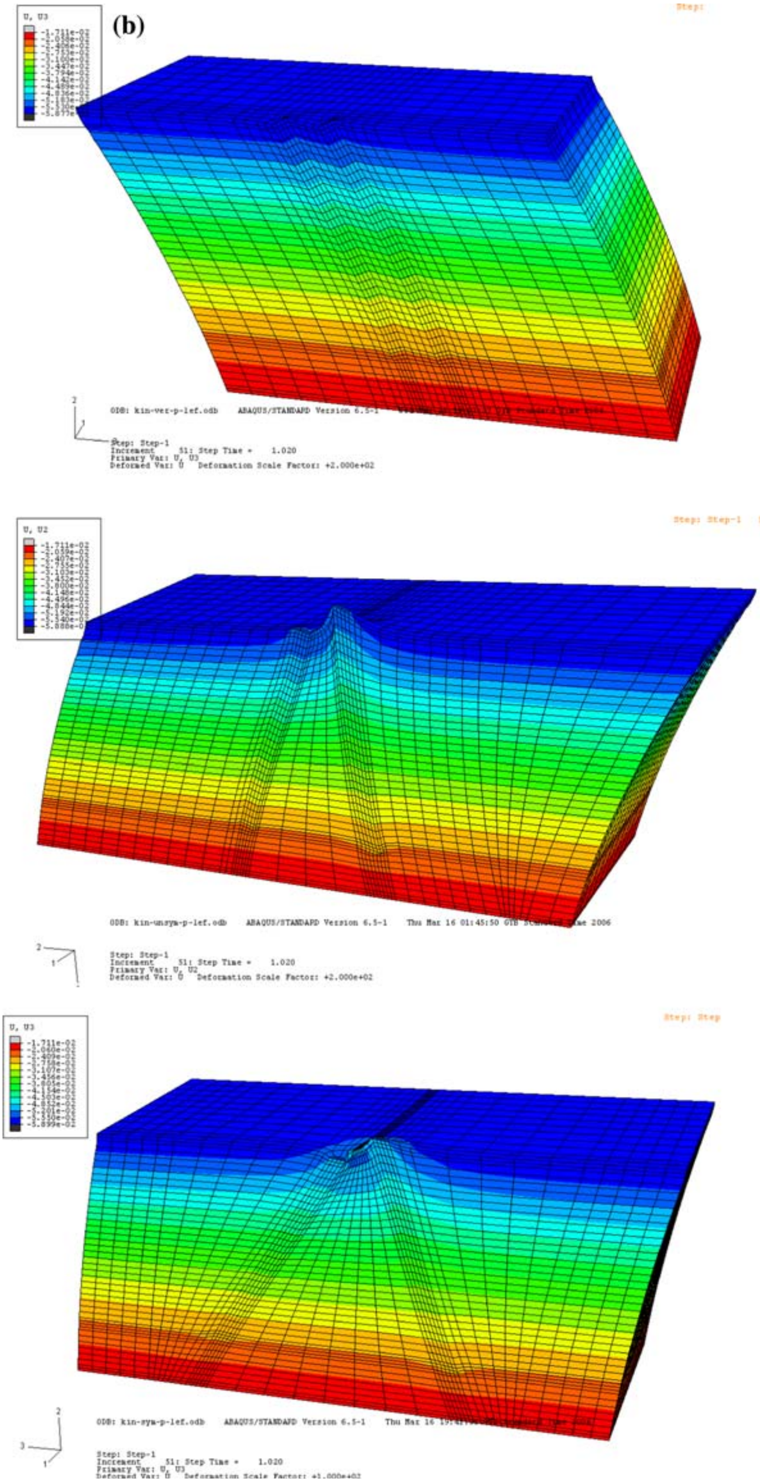

(b)
U, U3
ODB: kin-ver-p-lef.odb ABAQUS/STANDARD Version 6.5-1
Step: Step-1
Increment 51: Step Time = 1.020
Primary Var: U, U3
Deformed Var: U Deformation Scale Factor: +2.000e+02
U, U2
ODB: kin-unsym-p-lef.odb ABAQUS/STANDARD Version 6.5-1 Thu Mar 16 01:45:50 GTB Standard Time 2006
Step: Step-1
Increment 51: Step Time = 1.020
Primary Var: U, U2
Deformed Var: U Deformation Scale Factor: +2.000e+02
U, U3
ODB: kin-sym-p-lef.odb ABAQUS/STANDARD Version 6.5-1
Step: Step-1
Increment 51: Step Time = 1.020
Primary Var: U, U3

Fig. 5b

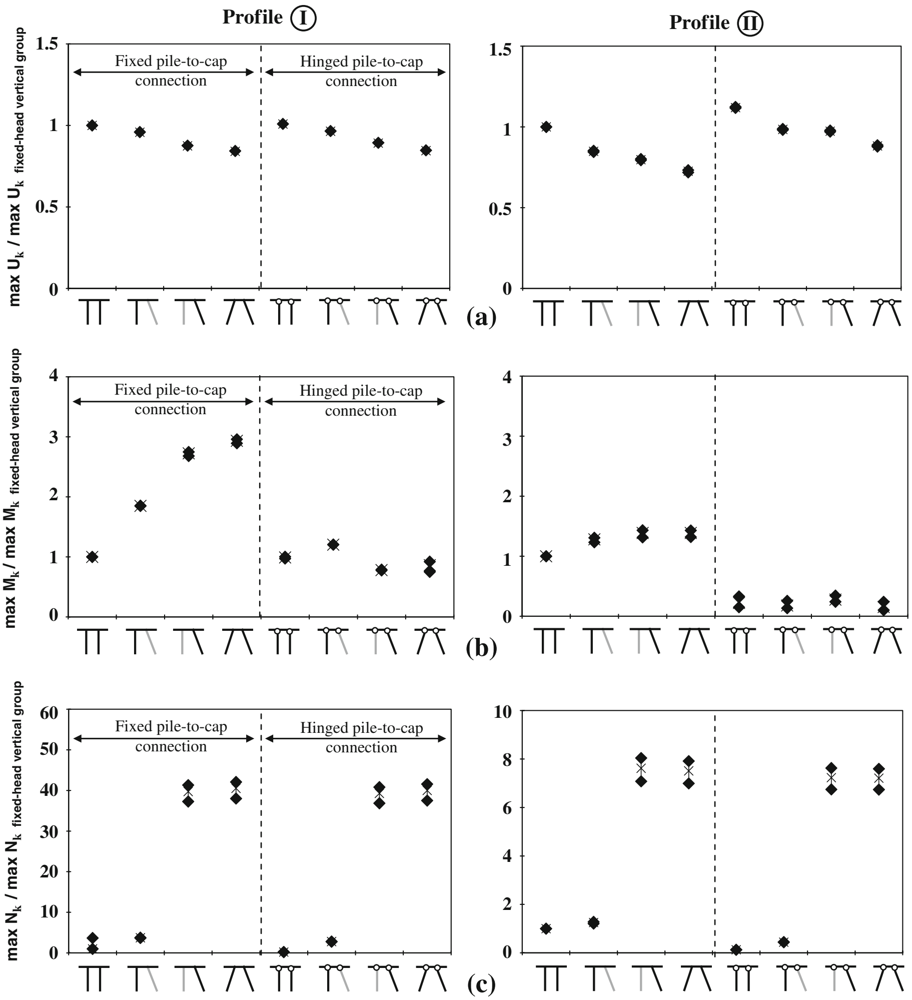


Fig. 6 Normalized maximum kinematic response: **a** peak value of maximum horizontal displacement, **b** peak value of maximum bending moment, and **c** peak value of maximum axial force along the pile. Normalization with respect to the response of the group of rigidly connected vertical piles embedded in a homogenous (Profile I, *left-hand column*) and a non-homogenous (Profile II, *right-hand column*) deposit. Maximum, minimum, and average values from the three accelerograms are shown as data points. The results refer to each configuration and the particular pile shown in *bold line* at the *bottom* of each figure

4.2 Soil–foundation–structure interaction

4.2.1 Slender superstructure: $H_{st} = 12\ m$

Results obtained with the tall superstructure (whose crusial loading is the overturning moment e.g. a bridge pier) are presented in Figs. 7, 8, and 9, for all earthquake records utilized. The following general conclusions are drawn:

- The symmetrically inclined-pile group in conjunction with hinged pile-to-cap connection results in the most satisfactory performance of both the superstructure and the foundation.

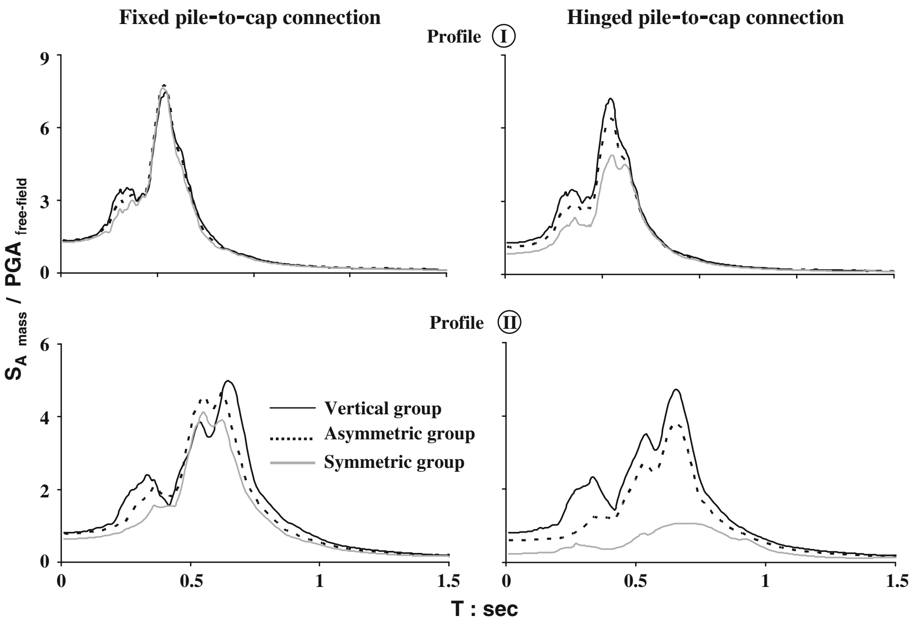


Fig. 7 Normalized "floor" response spectra of the three group configurations with respect to the free-field peak ground acceleration embedded in the homogenous (I) and the non-homogenous (II) soil profiles, for fixed and hinged pile-to-pile–cap connection. ($H_{\text{structure}} = 12$ m, excitation: Lefkada record)

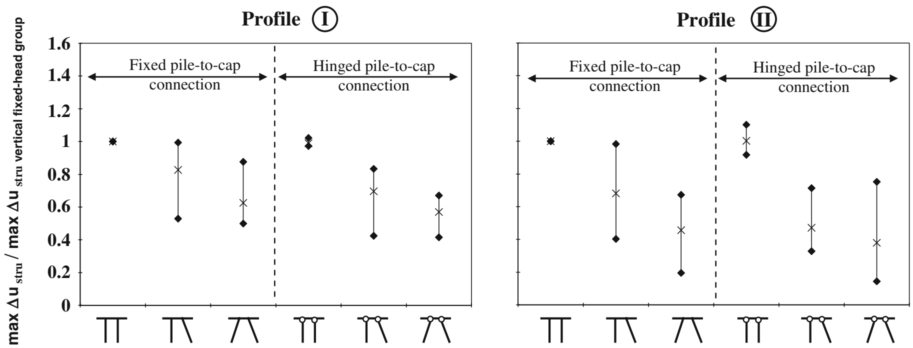


Fig. 8 Normalized maximum drift of the superstructure ($H_{\text{structure}} = 12$ m) with respect to its maximum counterpart of the group of rigidly connected vertical piles embedded in the homogenous (I, *left-hand column*) and the non-homogenous (II, *right-hand column*) soil profiles. Maximum, minimum, and average values from the three accelerograms are shown as data points. The results refer to each configuration shown at the *bottom* of the figure

The hinged connection results in a decrease of spectral acceleration values at the mass level (Fig. 7), and in smaller structural forces of the piles and the pier, compared to the group with vertical piles (Fig. 8). The horizontal drift, when the structure is supported on groups with inclined piles, is significantly smaller than for the vertical group for both types of pile-to-cap connection. This reduction of structural "distress" is attributed to the smaller pile–cap horizontal displacement, and to the larger cap rotations of the groups containing inclined piles.

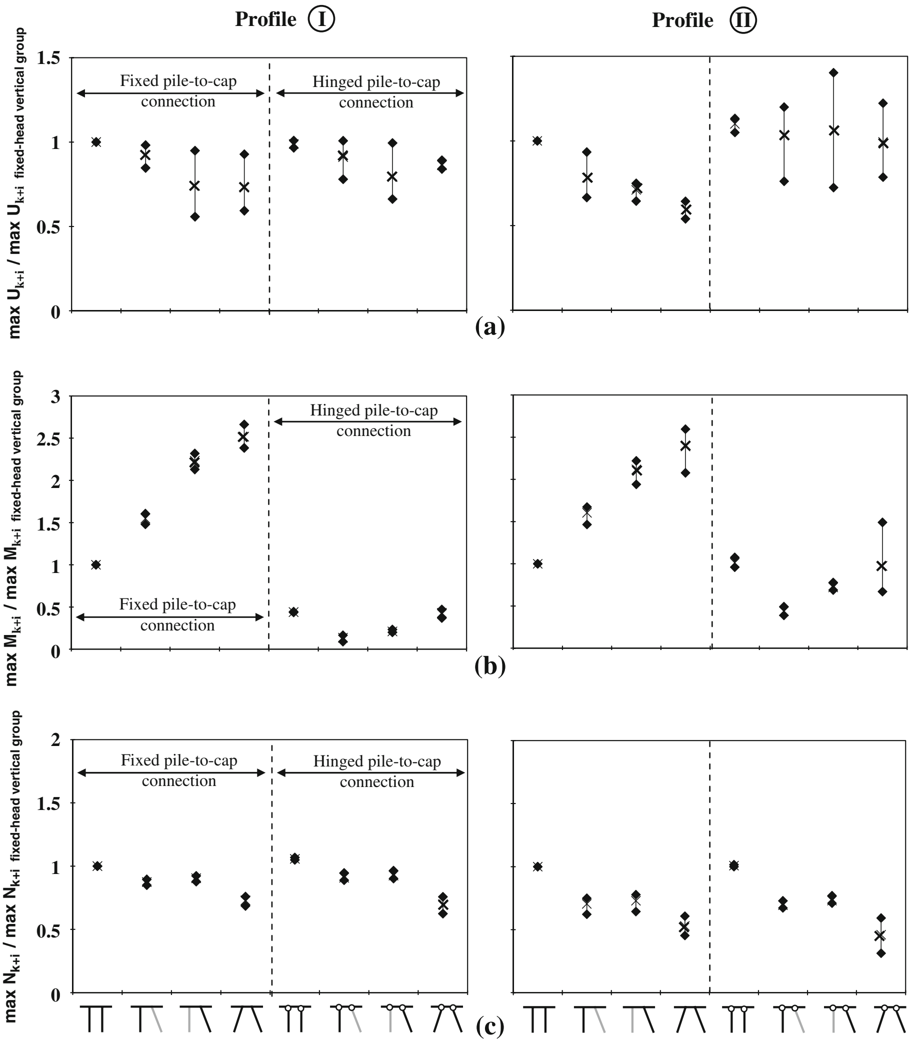


Fig. 9 Normalized maximum total (kinematic + inertial) response: (**a**) peak value of maximum horizontal displacement, (**b**) peak value of maximum bending moment, and (**c**) peak value of maximum axial force along the pile. Normalization with respect to the response of the group of rigidly connected vertical piles embedded in a homogenous (Profile I, *left-hand column*) and a non-homogenous (Profile II, *right-hand column*) deposit. Maximum, minimum, and average values from the three accelerograms are shown as data points. The results refer to each configuration and the particular pile shown in *bold line* at the *bottom* of each figure ($H_{\text{structure}} = 12$ m).

- Owing to their larger stiffness, configurations with inclined piles develop smaller lateral displacements than the vertical group (Fig. 9a). However, changing the degree of fixity to hinged condition at the pile-head leads to significant attenuation of this benefit.
- In both profiles, the bending moment that develops at the pile head for fixed pile-to-cap connection, when the group contains inclined piles, can be three times larger than the corresponding moment of a group with vertical piles (Fig. 9b). Proper reinforcement of the pile–cap connection is necessary for undertaking safely this bending moment and securing

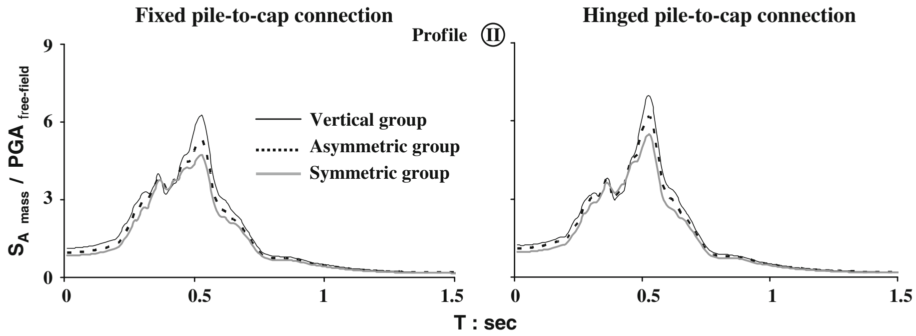


Fig. 10 Normalized "floor" response spectra of the three group configurations with respect to the free-field peak ground acceleration embedded in the non-homogenous (II) soil profile, for fixed and hinged pile-to-pile–cap connection ($H_{\text{structure}} = 1$ m, excitation: Lefkada record)

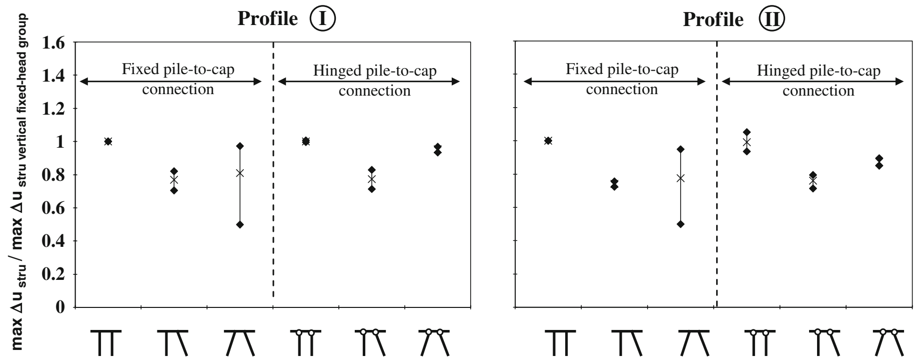


Fig. 11 Normalized maximum drift of the superstructure ($H_{\text{structure}} = 1$ m) with respect to its maximum counterpart of the group of rigidly connected vertical piles embedded in the homogenous (I, *left-hand column*) and the non-homogenous (II, *right-hand column*) soil profiles. Maximum, minimum, and average values from the three accelerograms are shown as data points. The results refer to each configuration shown at the *bottom* of the figure

adequate inelastic deformation in an unpredictably large earthquake motion. This is not a very easy task however, given the difficulty in achieving ductile behaviour of a connection with batter pile.

- Perhaps surprisingly, the symmetric group of inclined piles attracts smaller axial forces than those of the group with vertical piles (Fig. 9c)! This must be attributed to the disproportionately large overturning moment resisted mainly by axial reactions of the vertical piles. Naturally then, the fixity condition at the pile–cap does not influence significantly the magnitude of the axial force developed in the piles, especially with exclusively vertical piles in the group.

4.2.2 Short superstructure: $H_{\text{st}} = 1$ m

Results obtained with the short superstructure (whose crusial loading is the shear force, e.g. a quay-wall) are presented in Figs. 10, 11, and 12, for all earthquake records utilized. Moreover,

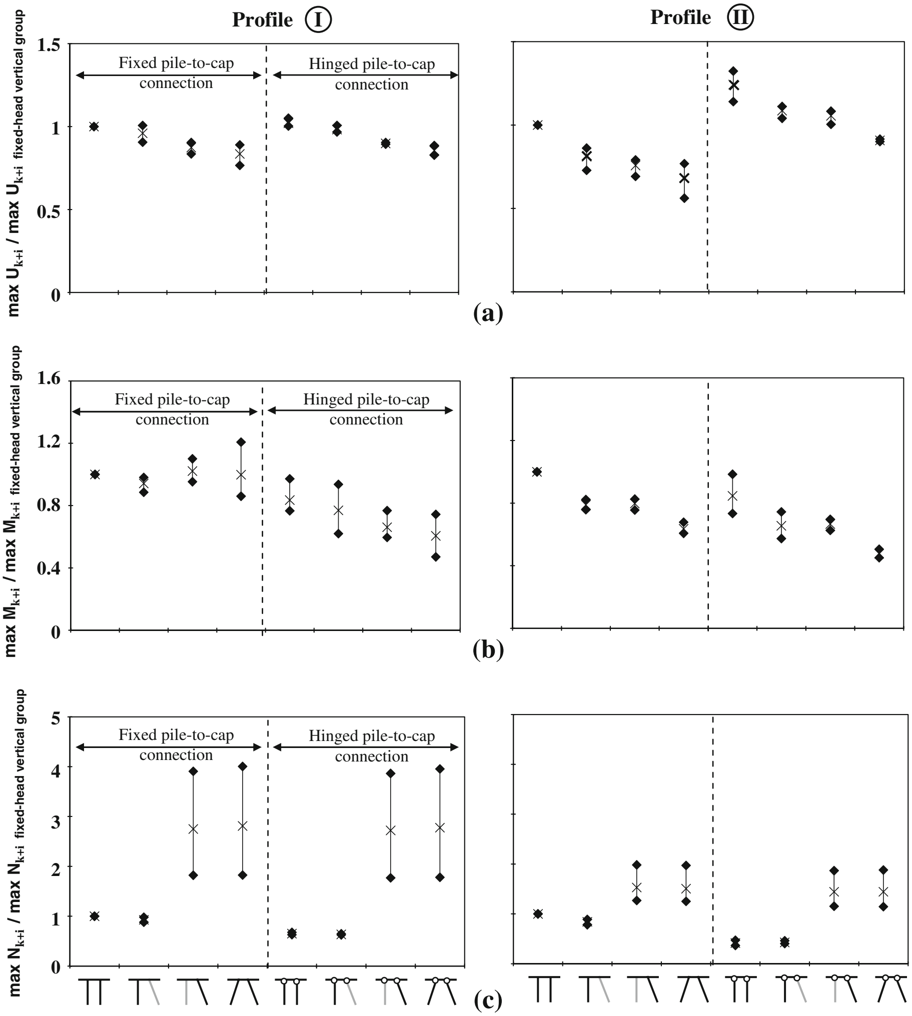


Fig. 12 Normalized maximum total (kinematic + inertial) response: (**a**) peak maximum horizontal displacement, (**b**) peak maximum bending moment, and (**c**) peak maximum axial force along the pile. Normalization with respect to the response of the group of rigidly connected vertical piles embedded in a homogenous (Profile I, *left-hand column*) and a non-homogenous (Profile II, *right-hand column*) deposit. Maximum, minimum, and average values from the three accelerograms are shown as data points. The results refer to each configuration and the particular pile shown in bold line at the bottom of each figure ($H_{\text{structure}} = 1$ m)

distributions of horizontal displacement and internal forces that develop along the pile at the time when the maximum occurs are depicted in Figs. 14 and 15 for both short and slender structure. The following conclusions are drawn:

- The type of pile-to-cap connection does not influence the response of the superstructure (Fig. 10), by contrast to the case of a very slender structure where the hinged pile-to-cap-connection significantly reduced the mass acceleration (Fig. 7).

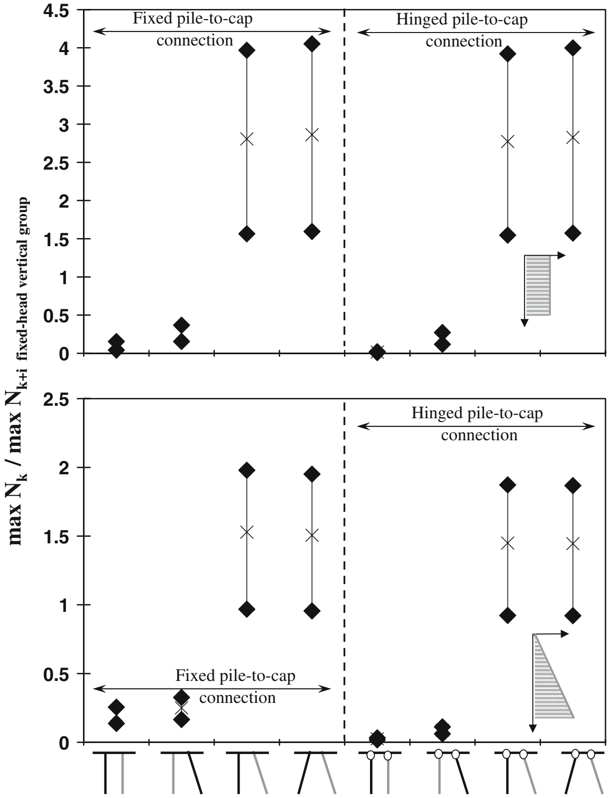


Fig. 13 Normalized peak maximum kinematic axial force along the pile. Normalization with respect to the total (kinematic + inertial) response of the group of rigidly connected vertical piles embedded in the homogenous (I), and the non-homogenous (II) soil profiles (maximum, minimum, and average values from the three accelerograms [Lefkada, Aegion, JMA]). The results refer to each configuration and the particular pile shown in *bold line* at the bottom of each figure ($H_{\text{structure}} = 1$ m)

- The horizontal drift of the superstructure is less sensitive to the type of foundation (symmetric or asymmetric, and fixed or hinged pile-to-cap connection), contrary to the behaviour of the slender structure (Fig. 11).
- Groups with inclined piles develop smaller bending moments and larger axial forces than the vertical group, for both soil profiles and both types of pile-to-cap connection (Fig. 12)—contrary to the case of tall superstructure. This observation is in agreement with published numerical and experimental studies (Sadek and Shahrour 2004; Okawa et al. 2005; Sadek and Shahrour 2006), and is also compatible with the prevailing engineering perception about the role of batter piles.
- Inclined piles develop systematically larger axial forces than the group with exclusively vertical piles. This maximum, however, occurs at great depths (in the order of 10 pile diameters) and is mainly due to the kinematic interaction of the pile with the soil as is also depicted in Fig. 13. In the case of short superstructure the kinematic component of response dominates upon the inertial one. In addition to lateral oscillation, inclined piles also develop oscillation along their longitudinal axis when subjected to a horizontally imposed base excitation which results in generation of axial forces. Kinematic-driven

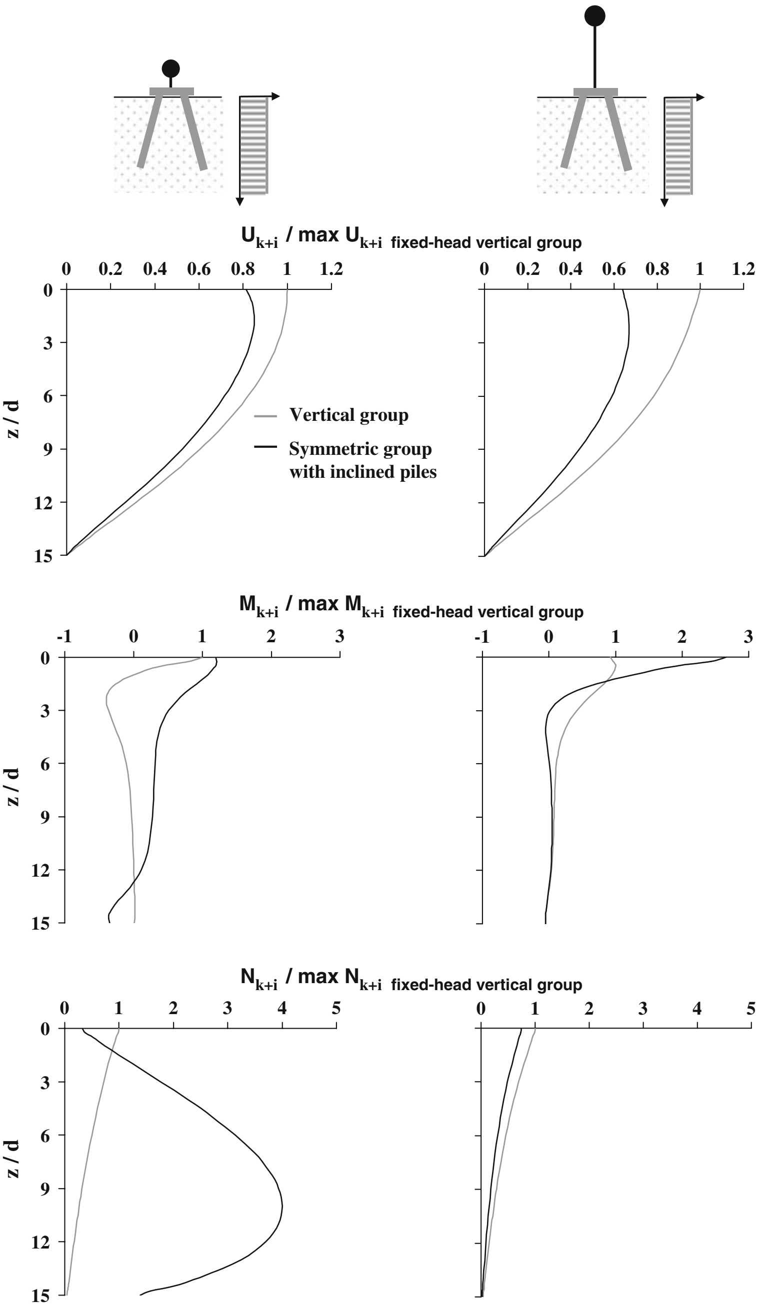


Fig. 14 Normalized distributions, for fixed pile-to-cap connection, of : **a** horizontal displacements, **b** bending moments, and **c** axial forces along the pile at the time when the maximum occurs in each configuration supporting a short (*left-hand column*) and a slender (*right-hand column*) structure. Normalization with respect to the response of the group of rigidly connected vertical piles embedded in a homogenous deposit (excitation: Lefkada record)

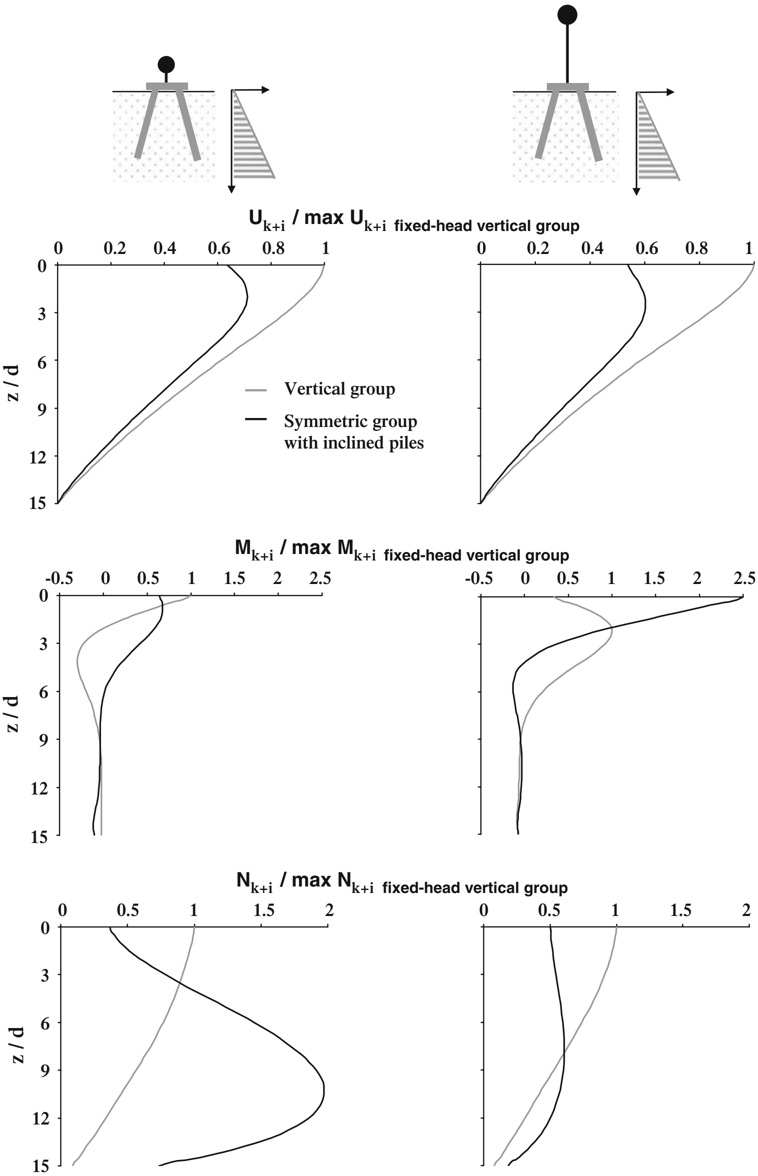


Fig. 15 Normalized distributions, for fixed pile-to-cap connection, of : **a** horizontal displacements, **b** bending moments, and **c** axial forces along the pile at the time when the maximum occurs in each configuration supporting a short (*left-hand column*) and a slender (*right-hand column*) structure. Normalization with respect to the response of the group of rigidly connected vertical piles embedded in a non-homogenous deposit (excitation: Lefkada record)

axial forces are vanishingly small in the group with exclusively vertical piles. In Fig. 13 the maximum value of only kinematic axial force along the piles in each configuration is normalized by the total (kinematic + inertial) axial force in the vertical fixed head pile group.

5 Conclusion

The paper has presented an extensive summary of the results of a more comprehensive parametric study on the seismic performance of inclined piles, conducted within the framework of the QUAKER project. Pile groups embedded in two soil profiles (a homogeneous and a non-homogeneous), with two pile-to-pile–cap connections (fixed and hinged), and subjected to three different acceleration time histories are studied. The results confirm that groups with batter piles provide large lateral stiffness. But, moreover, and perhaps surprisingly, for tall slender structures, such as a tall bridge pier, the group with symmetrically inclined piles (in conjunction with hinged pile-to-cap connection) results in the most satisfactory performance of both the superstructure and the foundation. It appears that the hinged connection acts as an imperfect seismic isolation not only for the foundation but also for the superstructure, resulting in smaller structural forces than in a group with exclusively vertical piles. On the contrary, in case of a short structure, the type of pile-to-cap connection only slightly influences the response of the superstructure. The aforementioned conclusions should be verified experimentally and should be strengthened by results from further analyses, for realistic soil profiles and nonlinear soil response.

Acknowledgments The work presented in this paper has been conducted with the financial support of the EU Fifth Framework Program: Environment, Energy and Sustainable Development Research and Technological Development Activity of Generic Nature: The fight against Natural and Technological Hazards, Research Project QUAKER, Contract number: EVG1-CT-2002-00064.

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