Proceedings of the 9th International Conference on Structural Dynamics, EURODYN 2014
Porto, Portugal, 30 June - 2 July 2014
A. Cunha, E. Caetano, P. Ribeiro, G. Müller (eds.)
ISSN: 2311-9020; ISBN: 978-972-752-165-4

Influence of pile rake angle on the seismic response of pile foundations and piled structures

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ABSTRACT: Even though inclined piles have traditionally been used to withstand large horizontal loads, the lack of understanding about their response under seismic loads (or under associated loads derived, for instance, from soil settlement or liquefaction) have prevented their use in seismic regions. However, several authors have showed that inclined piles may provide potential benefits with respect to the seismic response of the superstructure and of the foundation itself. For this reason, the dynamic response of this type of foundations is now being studied. The kinematic interaction factors of pile foundations with inclined elements is one of the aspects that have not received enough attention. The influence of pile rake angle on the seismic response of the superstructure is another aspect that still needs more research. These are precisely the two aspects on which this paper will focus. In order to compute kinematic interaction factors, a three-dimensional boundary element – finite element coupling formulation is used. The system is excited by harmonic vertically-incident shear waves. The paper presents kinematic interaction factors corresponding to inclined single piles, and square 2 by 2 and 3 by 3 pile groups in ready-to-use dimensionless format. It is shown that, while the kinematic response of inclined single piles is rather independent of the rake angle, the kinematic restriction of the pile cap produces a significant dependence on that factor: horizontal displacements are reduced with the rake angle, while rotation changes sign for a certain configuration and increases significantly afterwards. These results can be used to compute the response of a superstructure by substructuring in order to study further the influence of rake angle on it.

KEY WORDS: Pile Group; Battered Piles; Kinematic Interaction Factors; Soil-Structure Interaction.

1 INTRODUCTION

Inclined piles have traditionally been used in foundations when lateral resistance is required to transmit horizontal loads. However, the lack of understanding about their response under seismic loads have prevented their use in seismic regions and many codes require that such piles be avoided [1][2]. Nevertheless, several authors have showed that inclined piles may provide potential benefits with respect to the seismic response of the superstructure and of the foundation itself [3][4][5]. Furthermore, field evidences suggesting relevant beneficial effects concerning the use of raked piles has been recollected [6][7].

The beneficial or detrimental role of foundations including inclined piles has not been yet clarified. For this reason, the dynamic response of this type of foundations needs further research. The kinematic interaction factors of foundations consisting of vertical piles have been broadly studied [8][9][10][11][12][13]. However, the kinematic interaction factors of pile foundations with inclined elements have not received enough attention. On the other hand, the influence of pile rake angle on the seismic response of the superstructure is another aspect that still needs more research. This paper focuses on these two aspects and provide kinematic interaction factors needed to carry out substructuring analyses.

Some authors have performed analysis of the kinematic interaction factors of deep foundations with vertical piles (e. g. [14][15]). However, up to the authors' knowledge, only Giannakou [16] has presented kinematic interaction factors of inclined piles for groups of 2 x 1 piles.

In this line, this paper presents kinematic interaction factors of single raked piles, as well as those of 2 x 2 and 3 x 3 groups with battered elements. In order to obtain these results, a three-dimensional boundary element (BEM) – finite element (FEM) coupling formulation is used where the soil is modeled as a homogeneous viscoelastic isotropic half spaced by boundary elements, and the piles are modeled as Euler-Bernoulli beams, embedded in the soil, by monodimensional finite elements. Coupling is performed by equilibrium and compatibility conditions. The system is excited by harmonic vertically-incident shear waves. The main trends of the influence of the rake angle on the kinematic interaction factors are deduced from the presented results. Moreover, a procedure based on a substructuring methodology [20] is used herein to analyze the influence of the rake angle on the maximum response value of a superstructure supported on piles.

2 METHODOLOGY

2.1 *BEM-FEM model*

In this work, the kinematic interaction factors of pile foundations are numerically obtained by using a BEM-FEM coupling model [18][19].

The boundary element method is used herein to model the dynamic response of the soil region taking into account the internal loads arising from the pile-soil interaction. The piles rigidity is introduced later into the system by using finite elements. The whole approach is depicted in Figure 1.

The soil is considered as a linear, homogeneous, isotropic, viscoelastic half-space. The boundary integral equation for a time-harmonic elastodynamic state defined in this region with boundary Γ_s can be written in a condensed and general form as

$$\boldsymbol{c}^k\boldsymbol{u}^k + \int_{\Gamma_s} \boldsymbol{p}^*\boldsymbol{u}\, d\Gamma =$$

$$\int_{\Gamma_s} \boldsymbol{u}^*\boldsymbol{p}\, d\Gamma + \sum_{j=1}^{n_p}\left[\int_{\Gamma_{p_j}} \boldsymbol{u}^*\boldsymbol{q}^{s_j}\, d\Gamma_{p_j} + \Upsilon_j^k \boldsymbol{f}_{s_j}\right] \qquad (1)$$

where $\boldsymbol{c}^k$ is the local free term matrix at collocation point 'k', $\boldsymbol{u}$ and $\boldsymbol{p}$ represent the displacement and the traction fields in the three directions of space, $\boldsymbol{u}^*$ and $\boldsymbol{p}^*$ are the elastodynamic fundamental solution tensors due to a time-harmonic concentrated load at point 'k', n_p is the number of piles, and Γ_{p_j} represents the pile-soil interface along the load-line j. In Equation (1), the two terms between brackets represent the contribution of the internal loads, being $\boldsymbol{f}_{s_j}$ a point load placed at the tip of the pile and $\boldsymbol{q}^{s_j}$ the distribution of interaction loads, along the pile shaft, applied on a line defined by the pile axis. On the other hand, Υ_j^k represents the corresponding $\boldsymbol{u}^*$ tensor computed at the tip of the pile.

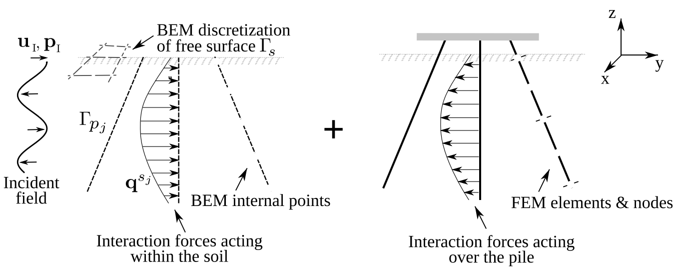

Figure 1. Boundary element-finite element model.

Following the usual procedure in the BEM, the numerical solution of Equation (1) requires the discretization of the boundary surface. Thus, the boundary surface Γ_s is discretized into quadratic elements of triangular and quadrilateral shape with six and nine nodes, respectively. Then, over each boundary element, displacement and traction fields $\boldsymbol{u}$ and $\boldsymbol{p}$ are approximated in terms of their values at nodal points making use of a set of polynomial interpolation functions [17]. Regarding the load lines and the evaluation of the last two terms of Equation (1), piles are discretized using three-node beam element where the distribution of tractions $\boldsymbol{q}^{s_j}$ is approximated, according to the corresponding pile finite-element discretization into nodes and elements and to the proper interpolation functions [18][19], in terms of its values $\overline{\boldsymbol{q}}^{s_j}$ defined at a series of internal nodes.

Now, Equation (1) can be written for all boundary nodes in Γ_s yielding the following matrix equation:

$$\boldsymbol{H}^{ss}\overline{\boldsymbol{u}} - \boldsymbol{G}^{ss}\overline{\boldsymbol{p}} - \sum_{j=1}^{n_p} \boldsymbol{G}^{sp_j}\,\overline{\boldsymbol{q}}^{s_j} - \sum_{j=1}^{n_p} \Upsilon_j^s\, f_{sj} = 0 \qquad (2)$$

where $\boldsymbol{H}$ and $\boldsymbol{G}$ are coefficient matrices obtained by numerical integration over the boundary elements of the fundamental solution times the corresponding shape functions, $\overline{\boldsymbol{u}}$ and $\overline{\boldsymbol{p}}$ are the vectors of nodal displacements and tractions of the boundary elements.

The wave field in the halfspace discretization ($\overline{\boldsymbol{u}}$) consists of two parts: the known incident field ($\overline{\boldsymbol{u}}_I$) and the unknown scattered field ($\overline{\boldsymbol{u}}_S$). The resulting displacement can be obtained by superposition as $\overline{\boldsymbol{u}} = \overline{\boldsymbol{u}}_I + \overline{\boldsymbol{u}}_S$. Thus, considering a pile foundation embedded in a soil subjected to incident waves, equation (2) can be written in terms of the scattered fields as

$$\boldsymbol{H}^{ss}\overline{\boldsymbol{u}} - \sum_{j=1}^{n_p} \boldsymbol{G}^{sp_j}\,\overline{\boldsymbol{q}}^{s_j} - \sum_{j=1}^{n_p} \Upsilon_j^s\, f_{sj} = \boldsymbol{H}^{ss}\overline{\boldsymbol{u}}_I \qquad (3)$$

where, taking into account the problem studied in this paper, the boundary conditions over the free surface Γ_s nodes ($\overline{\boldsymbol{p}}$=0) have been imposed.

Equation (1) can be also applied on internal nodes belonging to load-line Γ_{p_i}, yielding to the following equation

$$\overline{\boldsymbol{u}}^{p_i} + \boldsymbol{H}^{p_i s}\overline{\boldsymbol{u}} - \sum_{j=1}^{n_p} \boldsymbol{G}^{p_i p_j}\overline{\boldsymbol{q}}^{s_j} - \sum_{j=1}^{n_p} \Upsilon_j^{\,p_i} \boldsymbol{f}_{s_j} = \overline{\boldsymbol{u}}_I^{p_i} + \boldsymbol{H}^{p_i s}\overline{\boldsymbol{u}}_I \qquad (4)$$

where $\overline{\boldsymbol{u}}^{p_i}$ is the vector of nodal displacements along load-line i.

The dynamic behavior of pile j in a finite element sense, can be described as

$$\left(\boldsymbol{K}_j - \boldsymbol{\omega}^2\boldsymbol{M}_j\right)\overline{\boldsymbol{u}}_j^p = \boldsymbol{f}_j^{ext} - \boldsymbol{Q}_j\overline{\boldsymbol{q}}^{s_j} \qquad (5)$$

where $\overline{\boldsymbol{u}}_j^p$ is the vector of nodal translation and rotation amplitudes along the pile, $\boldsymbol{f}_j^{ext}$ represents the punctual forces acting at the top and the tip of the pile and $\boldsymbol{Q}_j$ is the matrix that transforms the nodal tractions to equivalent nodal forces. $\boldsymbol{M}$ and $\boldsymbol{K}$ are the mass and stiffness matrices of the pile, respectively.

Imposing additional equations of equilibrium and compatibility by correlating BEM load lines and FEM piles, Equations (3), (4) and (5) can be rearranged in a system of equations representing the soil-pile foundation problem.

2.2 Substructuring methodology

A procedure based on a substructuring methodology [20] is used herein to analyze the influence of the rake angle on the maximum response value of a superstructure supported on piles. The system is subdivided into building-cap structure and soil-foundation stiffness and damping, represented by springs and dashpots. The solution is broken into three steps, as proposed by Kausel and Roësset [21]. The determination of the horizontal and rocking motions of the massless pile cap constitutes the first step. The second step is to obtain the impedances. In this study, kinematic interaction factors and impedance functions are computed by the BEM-FEM coupling model described in Section 2.1. Finally, the response of the structure supported on springs and subjected to the motion computed in the first step is obtained at each frequency.

3 RESULTS

3.1 Problem description

The configurations of all the foundations under investigation consist of square regular groups of piles which are symmetrical with respect to planes *xy* and *yz*. All piles have identical material and geometrical properties. Pile heads are constrained by a rigid pile-cap which is not in contact with the half-space. The main geometrical parameters of the system (see Figure 2) are: piles length (L) and diameter (d), spacing between centers of adjacent pile heads (s), and rake angle between the vertical and the pile axis (θ). The foundation halfwidth is defined as $b=d$ for single piles, $b=s$ for 2 x 2 pile groups, and $b=3s/2$ for 3 x 3 pile groups. In this work, the following dimensionless parameters are considered: pile spacing ratio s/d, pile-soil Young's modulus ratio E_p/E_s, soil-pile density ratio ρ_s/ρ_p, pile slenderness ratio L/d, soil Poisson's ratio ν_s, soil internal hysteretic damping coefficient β_s and dimensionless excitation frequency ω. In the latter expression c_s is the speed of propagation of shear waves in the half-space.

Translational and rotational kinematic interaction factors I_u and I_φ are both frequency dependent functions that can be defined as the horizontal (u_g) and rocking (φ_g) motions measured at the pile cap level and normalized with the free-field motion at the surface (u_{g_o}).

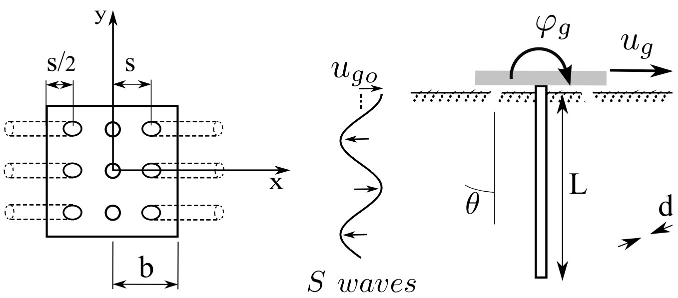

Figure 2. Foundation geometry.

A model consisting of a single-degree-of-freedom system in its fixed-base condition is used herein to study the dynamic behavior of linear shear structures (see Figure 3). The columns of the structure are supposed to be massless and axially inextensible. Both the foundations mass and the structural mass are assumed to be uniformly distributed over square areas. The dynamic behavior of the structure can be defined by its fixed-base fundamental period T, the height h of the resultant of the inertia forces for the first mode, the mass m participating in this mode, the moment of inertia of the vibrating mass I, the structural stiffness, and the viscous damping ratio ξ.

The system response can be approximated by that of a three-degree-of-freedom system defined by the structural horizontal deflection u and the foundation horizontal displacement u^c and rocking φ^c.

In this study, the dynamic response of the structure is obtained by means of an equivalent viscously damped single-degree-of-freedom (SDOF) oscillator whose dynamic characteristics have been previously determined by a simplified and accurate procedure presented in [20]. The transfer function used to establish this equivalence is the ratio of the shear force at the base of the structure to the effective earthquake force Q

$$Q = \left|\frac{\omega_n^2 u}{\omega^2 u_{g_o}}\right| \tag{6}$$

where $\omega_n^2 = 2\pi/\tilde{T}$, being $\tilde{T}$ the undamped natural period of the SDOF equivalent system.

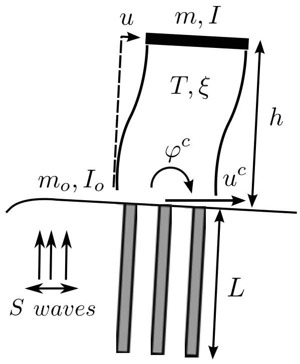

Figure 3. Soil-foundation-structure system.

In order to characterize the soil-foundation-structure system, a set of dimensionless parameters, covering the main features of SSI problems, has been used. These include, among others, the wave parameter $\sigma = c_s T/h$ measuring the soil-structure relative stiffness, the structural slenderness ratio h/b and the mass density ratio δ between structure and supporting soil.

It is assumed that $\delta = 0.15$, $m_o/m = 0$, $\xi = 0.05$, $h/b = 1, 10$ and $0 < 1/\sigma < 0.5$.

3.2 Kinematic interaction factors

This section presents kinematic interaction factors of single inclined piles, as well as 2 x 2 and 3 x 3 pile groups including battered elements, according to the description exposed in Section 3.1 and subjected to vertically-incident plane shear S waves. The following properties are assumed: β_s=0.05, ν_s=0.4, ρ_s/ρ_p=0.7 and $E_p/E_s=10^3$. Three different rake angles have been considered: θ=10º, 20º and 30º. This work comprises results corresponding to pile groups with piles inclined parallel or perpendicular to the direction of excitation. In order to maintain symmetry with respect to planes *xz* and *yz*, some vertical piles are included in 3 x 3 pile groups.

Figure 4 shows the kinematic interaction factors of a free-head single pile inclined perpendicular to the direction of excitation. It can be seen that inclining the pile in this direction leads to increasing values of the horizontal motion in the intermediate-frequency region, which is considered as a detrimental behavior. No beneficial effects can be observed in the low-frequency range. The rocking motion at the pile cap slightly increases with the rake angle for mid-to-high frequencies. However, in the low-frequency range, rotation slightly increases for decreasing rake angles.

Figure 5 depicts the kinematic interaction factors of a free-head single pile inclined parallel to the direction of excitation. In this case, increasing rake angles lead to lower values of the horizontal motion for dimensionless frequencies lower than 0.5. Regarding the rotational kinematic interaction factor, it can be seen that there are no significant differences with respect to the results shown in Figure 4 indicating that, in the case of single piles, this factor is almost independent of the direction of inclination.

Figure 6 and Figure 8 illustrate the influence of the rake angle on the kinematic interaction factors of a 2 x 2 and a 3 x 3 pile group, respectively, with piles inclined perpendicular to the direction of excitation. In both cases, it can be seen that inclining piles in this direction leads generally to higher values of the translational kinematic interaction factor in comparison with those obtained with vertical piles. A modest increase of rotation with the rake angle is observed in the intermediate frequency range.

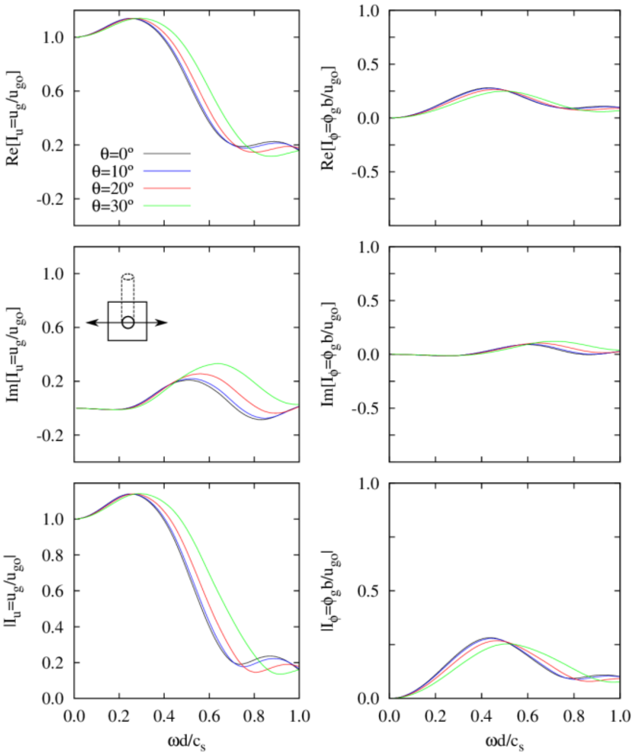

Figure 4. Kinematic interaction factors of a single pile inclined perpendicular to the direction of excitation with different rake angles θ, L/d=15 and L/b=15. $E_p/E_s=10^3$.

Figure 7 and Figure 9 shows the influence of the rake angle on the kinematic interaction factors of a 2 x 2 and a 3 x 3 pile group, respectively, with piles inclined parallel to the direction of excitation. The use of piles inclined in this direction results in a reduction of the horizontal motion in the low-to-mid frequency range. However, rotation increases with the rake angle. It should be noted that when pile groups include piles inclined parallel to the direction of excitation, horizontal free-field ground motion and cap rotation become out of phase.

Figure 10 shows the rotational kinematic interaction factor of a 3 x 3 pile group with piles inclined parallel to the direction of excitation for two different values of the pile-soil Young's modulus ratio. Opposite to what occurs with vertical piles, higher stiffness ratios E_p/E_s yields smaller cap rotations.

In order to illustrate the influence of the pile slenderness ratio on the rotation at pile cap, Figure 11 depicts the rotational kinematic interaction factor of a 3 x 3 pile group with piles inclined parallel to the direction of excitation for three different values of L/d. It can be seen that, in contrast to what occurs for vertical piles, lower pile slenderness ratios lead to decreasing rocking motions at the pile cap in a low-frequency range.

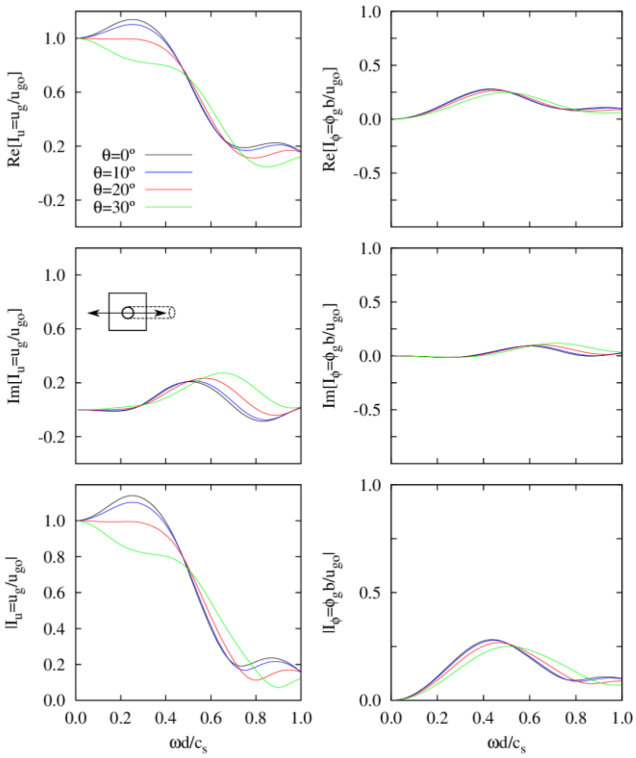

Figure 5. Kinematic interaction factors of single piles inclined parallel to the direction of excitation with different rake angles θ, L/d=15 and L/b=15. $E_p/E_s=10^3$.

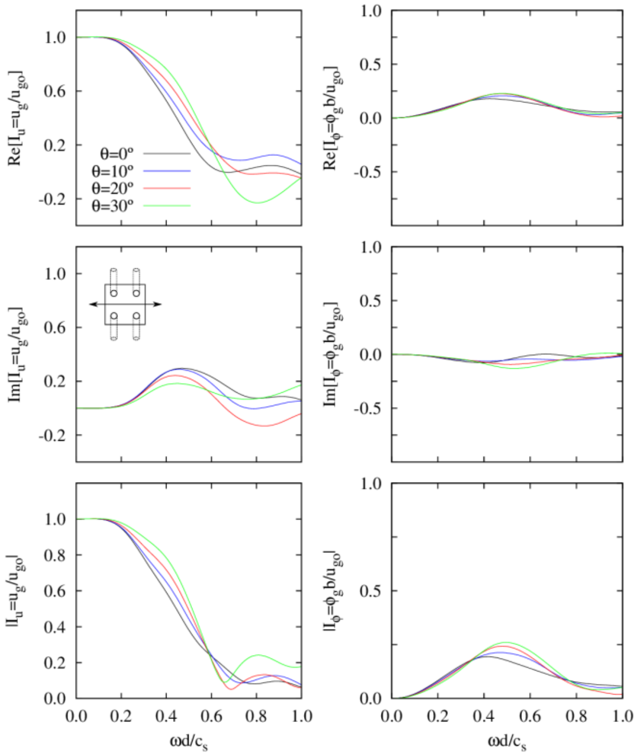

Figure 6. Kinematic interaction factors of 2 x 2 pile groups with piles inclined perpendicular to the direction of excitation with different rake angles θ, s/d=7.5, L/d=15 and L/b=2. $E_p/E_s=10^3$.

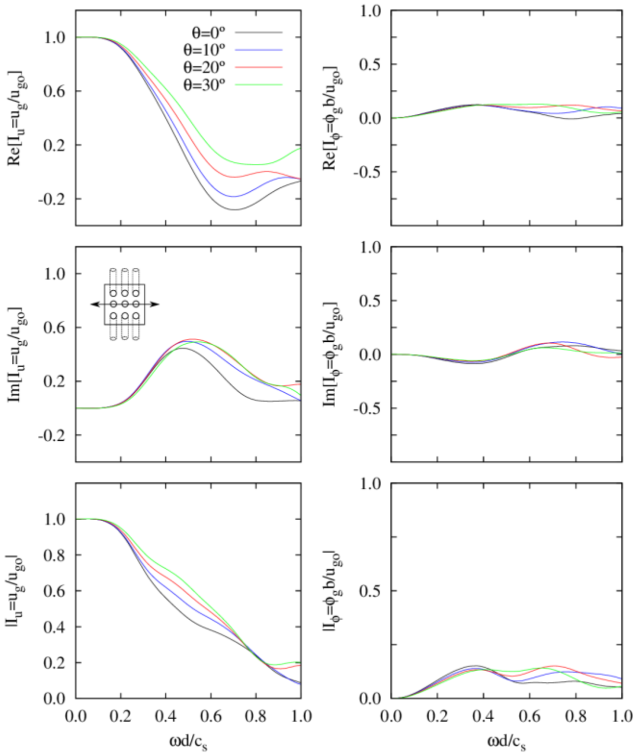

Figure 8. Kinematic interaction factors of 3 x 3 pile groups with piles inclined perpendicular to the direction of excitation with different rake angles θ, s/d=5, L/d=15 and L/b=2. $E_p/E_s=10^3$.

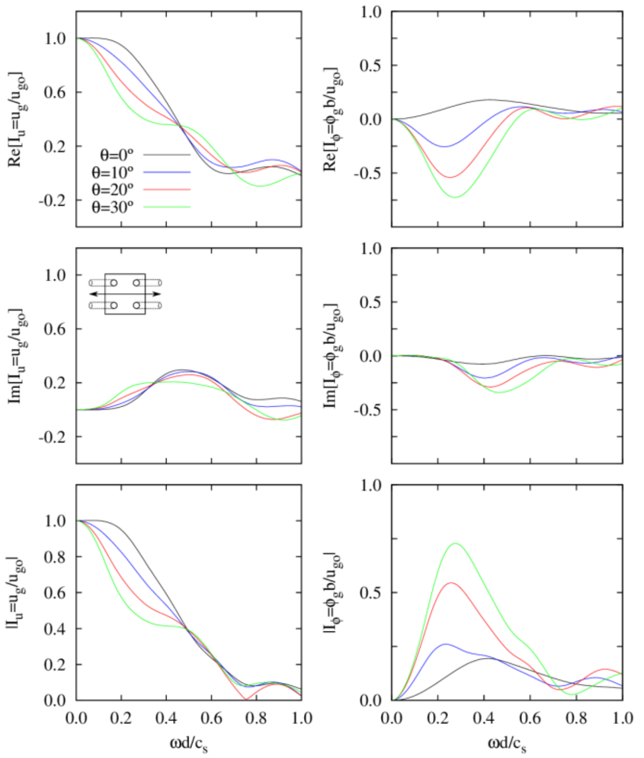

Figure 7. Kinematic interaction factors of 2 x 2 pile groups with piles inclined parallel to the direction of excitation with different rake angles θ, s/d=7.5, L/d=15 and L/b=15. $E_p/E_s=10^3$.

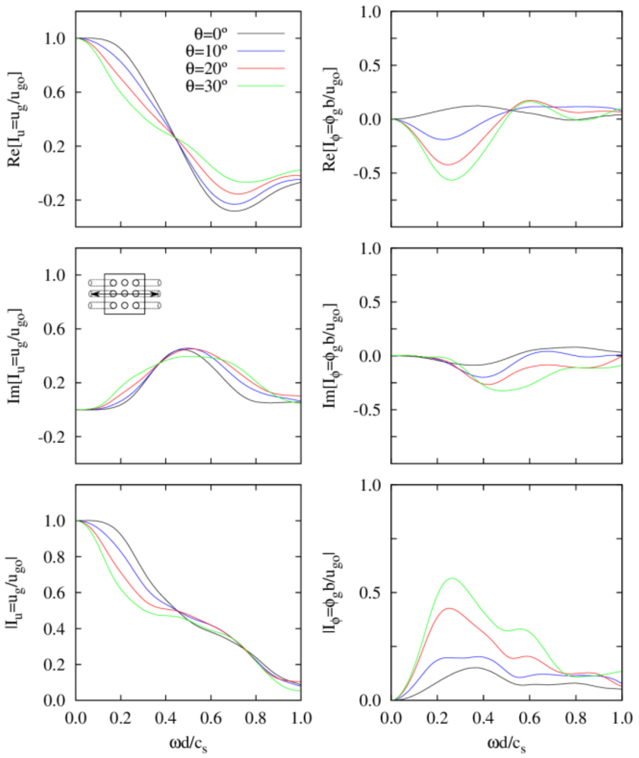

Figure 9. Kinematic interaction factors of 3 x 3 pile groups with piles inclined parallel to the direction of excitation with different rake angles θ, s/d=5, L/d=15 and L/b=2. $E_p/E_s=10^3$.

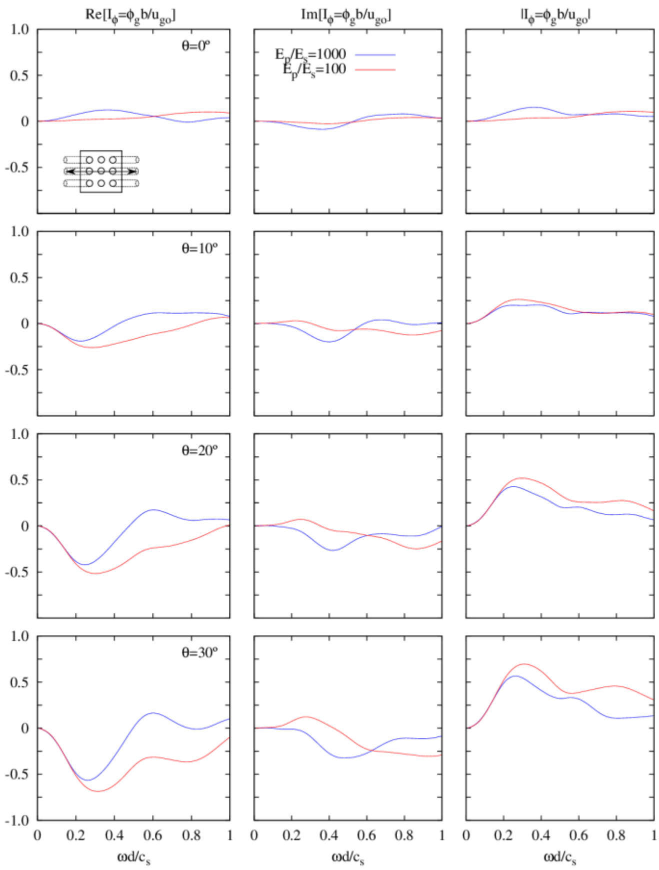

Figure 10. Influence of the pile-soil modulus ratio E_p/E_s on the rotational kinematic interaction factor of a 3 x 3 pile group with s/d=5, L/d=15, L/b=2 and piles inclined parallel to the direction of excitation.

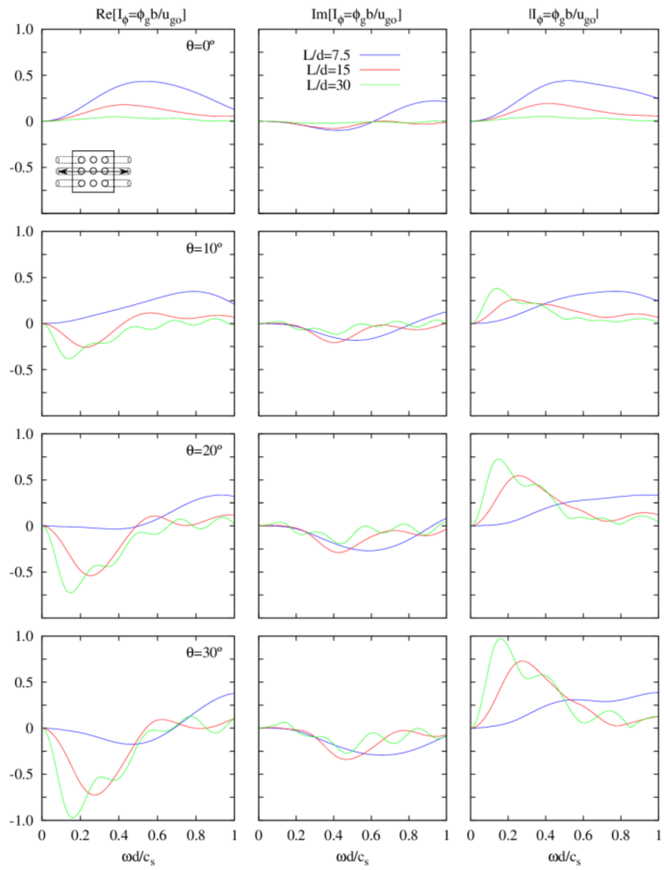

Figure 11. Influence of the pile slenderness ratio L/d on the rotational kinematic interaction factor of a 3 x 3 pile group with L/b=2 and piles inclined parallel to the direction of excitation. $E_p/E_s=10^3$.

3.3 *Maximum structural shear forces*

Figure 12 illustrates the influence of the rake angle on the maximum structural response value for a 2 x 2 pile group with piles inclined parallel to the direction of excitation. It should be noted that, for short and squat buildings with h/b=1, the value of Q_m decreases as the rake angle θ increases due to the reduction of the translational kinematic interaction factor, which predominates in this case. However, for high buildings with h/b=10, the controlling factor is that associated to rotation which increases with the rake angle and consequently leads to higher values of Q_m.

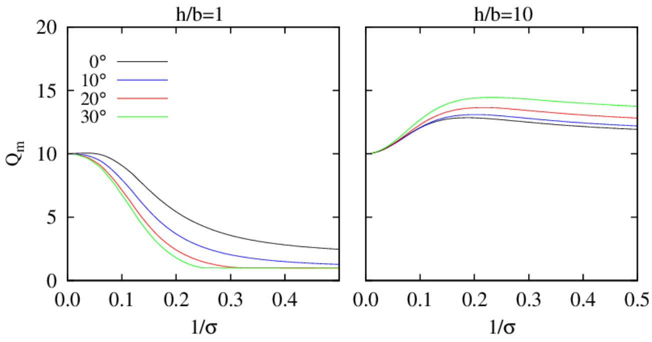

Figure 12. Maximum structural response value Q_m for 2 x 2 pile groups with piles inclined parallel to the direction of excitation with different rake angles θ, s/d=7.5, L/d=15 and L/b=2. $E_p/E_s=10^3$.

4 CONCLUSIONS

In this paper, an analysis of the influence of using pile groups with inclined piles, in two different directions and with three different rake angles, is accomplished. For this purpose, kinematic interaction factors of single piles, as well as 2 x 2 and 3 x 3 pile groups has been obtained using a boundary element-finite element methodology in which piles have been considered to be embedded in a homogeneous half-space and subjected to vertically incident plane shear S waves.

The main conclusions extracted from the analysis of the results shown in Section 3 are summarized below.

- The influence of the rake angle on the kinematic interaction factors strongly depends on the direction of inclination of piles except in the case of single battered piles where the rotational kinematic interaction factor is almost independent of pile inclination.
- Deep foundations including piles inclined parallel to the direction of excitation have a beneficial role in the low-to-mid frequency range leading to lower values of the horizontal displacement. The opposite occurs when piles are inclined perpendicular to the direction of excitation.
- Cap rotation increases with the rake angle and becomes out of phase with the horizontal free-field ground motion when inclining piles parallel to the direction of excitation.
- Opposite to what occurs with vertical piles, higher stiffness ratios E_p/E_s yields smaller cap rotations when considering configurations with elements inclined parallel to the direction of excitation.
- In contrast to what occurs for vertical piles, when considering deep foundations including piles inclined parallel to the direction of excitation, lower pile slenderness ratios lead to decreasing values of the rocking motion at the pile cap in a low-frequency range.
- The influence of the rake angle on the maximum structural shear force depends on the structural slenderness ratio. For high buildings (h/b=10) it increases with the rake angle. The opposite occurs for short and squat building (h/b=1).

Results in terms of translational and rotational kinematic interaction factors are provided for the purpose of allowing to compute the response of a superstructure by substructuring in order to study further the influence of rake angle on it.

ACKNOWLEDGMENTS

This work was supported by the Subdirección General de proyectos de Investigación of the Ministerio de Economía y Competitividad (MINECO) of Spain and FEDER through research project BIA2010-21399-C02-01 and also by the Agencia Canaria de Investigación, Innovación y Sociedad de la información (ACIISI) of the Government of the Canary Islands and FEDER through research project ProID20100224. Cristina Medina is a recipient of a fellowship from de Program of predoctoral fellowships of the University of Las Palmas de Gran Canaria (ULPGC). The authors are grateful for this support.

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