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BOOK OF ABSTRACTS

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ABSTRACT

It is well known that the seismic response of structures is highly dependent on the flexibility of the soil-foundation system and on the kinematic interaction between the foundation and the incident wave field. It is also accepted that through-the-soil interaction modifies the behaviour of nearby foundations under seismic excitation and, consequently, the seismic response of a structure may be significantly influenced by the presence of other structures. For this reason, the risk associated with the grouping of buildings should be assessed. Some interesting works have been done in this direction [1-4], but the problem has probably not received enough attention from the research community. For instance, to the extent of the authors' knowledge, no study assessing the dynamic through-the-soil interaction between adjacent piled structures under seismic excitation has been reported to date.

For this reason, a previously developed 3D BEM-FEM coupling model [5] for the dynamic analysis of pile foundations, where the Boundary Element Method (BEM) is used to model the soil, and the Finite Element Method (FEM) is used to model the piles as Euler-Bernoulli beams, has been enhanced to include the presence of piled structures (modeled by FEM) made up by vertical extensible piers and horizontal rigid slabs. The resulting code allows the analysis in the frequency domain of the dynamic behavior of groups of structures, three-dimensionally arranged, founded on multilayered viscoelastic soils through one or more pile caps.

As a first step, and in order to focus on structure-soil-structure interaction effects, results are presented for groups of buildings modeled as one-storey shear structures founded on 3×3 pile groups on a viscoelastic halfspace. The dynamic behavior of different configurations of structures subjected to S and Rayleigh waves is analyzed. It is shown that through-the-soil interaction between structures of similar dynamic properties affects the system response, mainly around its fundamental frequency. The seismic response of any of the structures can either increase or decrease in presence of other structures depending on the distance between adjacent buildings, i.e., there are values of this distance for which the seismic response of the system is amplified, but there are other values for which the response is attenuated, so that the structural risk diminishes in case of a seismic event.

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DYNAMIC THROUGH–THE–SOIL INTERACTION BETWEEN ADJACENT PILED STRUCTURES BY BEM-FEM MODEL

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Keywords: Soil–structure interaction (SSI), Structure–soil–structure interaction (SSSI), Pile, Seismic response, Boundary elements, BEM–FEM coupling

Abstract. *Not only the dynamic response of structures is highly dependent on the flexibility of the soil-foundation system and on the kinematic interaction between the foundation and the incident wave field, but it also may be significantly influenced by the presence of neighbouring structures by means of through-the-soil interaction.*

To address this problem in the case of adjacent piled structures, a previously developed 3D BEM-FEM coupling model for the dynamic analysis of pile foundations, where the Boundary Element Method (BEM) is used to model the soil, and the Finite Element Method (FEM) is used to model the piles as Euler-Bernoulli beams, has been enhanced to include the presence of piled structures (modeled by FEM) made up by vertical extensible piers and horizontal rigid slabs. The resulting code allows the analysis in the frequency domain of the dynamic behaviour of groups of structures, three-dimensionally arranged, founded on multilayered viscoelastic soils through one or more pile caps.

This way, the dynamic behaviour of different configurations of structures subjected to S and Rayleigh waves is analysed in this paper. As a first step, the structures have been modeled as one-storey shear structures founded on 3$\times$3 pile groups on a viscoelastic halfspace. It is shown that through-the-soil interaction between structures of similar dynamic properties affects the system response, mainly around its fundamental frequency. The seismic response of any of the structures can either increase or decrease in presence of other structures depending on the distance between adjacent buildings, i.e., there are values of this distance for which the seismic response of the system is amplified, but there are other values for which the response is attenuated, so that the structural risk diminishes in case of a seismic event.

1 INTRODUCTION

It is well known that the seismic response of structures is highly dependent on the flexibility of the soil-foundation system and on the kinematic interaction between the foundation and the incident wave field [1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12]. However, it is also accepted that through-the-soil interaction modifies the behaviour of nearby foundations under seismic excitation and, consequently, the seismic response of a structure may be significantly influenced by the presence of other structures. For this reason, the risk associated with the grouping of buildings should be assessed. A number of interesting works have been done in this direction [13, 14, 15, 16, 17, 18, 19, 20], but the problem has probably not received enough attention from the research community. For instance, to the extent of the authors' knowledge, no study assessing the dynamic through-the-soil interaction between adjacent piled structures under seismic excitation has been reported to date.

For this reason, a previously developed 3D BEM-FEM coupling model for the dynamic analysis of pile foundations [21, 22], where the Boundary Element Method (BEM) is used to model the soil, and the Finite Element Method (FEM) is used to model the piles as Euler-Bernoulli beams, has been enhanced to include the presence of piled structures (modeled by FEM) made up by vertical extensible piers and horizontal rigid slabs. The resulting code allows the analysis in the frequency domain of the dynamic behaviour of groups of structures, three-dimensionally arranged, founded on multilayered viscoelastic soils through one or more pile caps. Fig. 1 sketches the problem at hand.

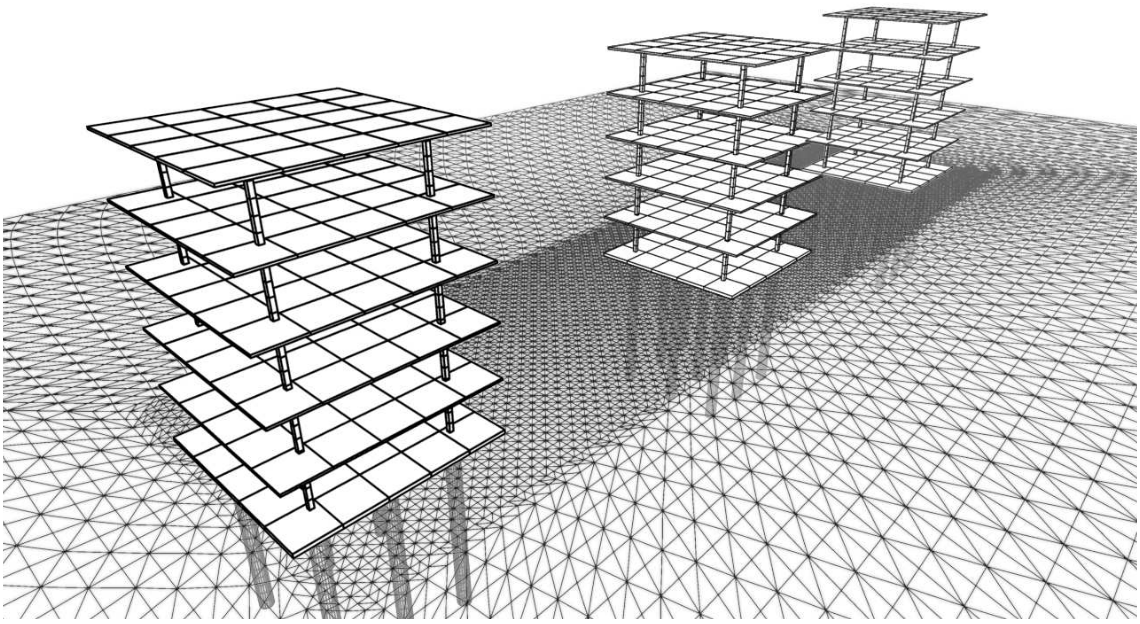

Figure 1: Group of neighbouring pile supported buildings.

As a first step, and in order to focus on structure-soil-structure interaction effects, results are presented for groups of buildings modeled as one-storey shear structures founded on 3×3 pile groups on a viscoelastic halfspace. The dynamic behaviour of different configurations of structures subjected to S and Rayleigh waves is analysed. It is shown that through-the-soil interaction between structures of similar dynamic properties affects the system response, mainly around its fundamental frequency. The seismic response of any of the structures can either increase or decrease in presence of other structures depending on the distance between adjacent buildings, i.e., there are values of this distance for which the seismic response of the system is amplified, but there are other values for which the response is attenuated, so that the structural risk diminishes in case of a seismic event.

2 BEM–FEM model

A boundary elements – finite elements coupling scheme has been used to obtain the results shown in this communications. In such formulation, each stratum of the soil is modelled by the BEM as a linear, homogeneous, isotropic, viscoelastic, unbounded region with complex valued shear modulus μ of the type $\mu = Re[\mu](1 + 2\mathrm{i}\beta)$, where β is the damping coefficient. The boundary integral equation for a time-harmonic elastodynamic state defined in a domain Ω_m with boundary Γ^m can be written in a condensed and general form as

$$\mathbf{c}^\iota \mathbf{u}^\iota + \int_{\Gamma^m} \mathbf{p}^* \mathbf{u}\, d\Gamma = \int_{\Gamma^m} \mathbf{u}^* \mathbf{p}\, d\Gamma + \int_{\Omega_m} \mathbf{u}^* \mathbf{X}\, d\Omega \tag{1}$$

where $\mathbf{c}^\iota$ is the local free term matrix at collocation point $\mathbf{x}^\iota$, $\mathbf{X}$ are the body forces in the domain Ω_m, $\mathbf{u}$ and $\mathbf{p}$ are the displacement and traction vectors, and $\mathbf{u}^*$ and $\mathbf{p}^*$ are the elastodynamic fundamental solution tensors representing the response of an unbounded region to a harmonic concentrated unit load with a time variation $e^{\mathrm{i}\omega t}$ applied at a point $\mathbf{x}^\iota$.

Generally, body forces $\mathbf{X}$ are considered to be zero in most of the elastodynamic problems. Nevertheless, in this approach, from the integral equation point of view, the pile-soil interaction takes place through internal punctual forces placed at the geometric piles tip and through load-lines placed along the piles axis, as it is assumed that the soil continuity is not altered by the presence of the piles. Under these assumptions, eq. (1) can be written as

$$\mathbf{c}^\iota \mathbf{u}^\iota + \int_{\Gamma^m} \mathbf{p}^* \mathbf{u}\, d\Gamma = \int_{\Gamma^m} \mathbf{u}^* \mathbf{p}\, d\Gamma + \sum_{j=1}^{n_{ll}^m} \left[\int_{\Gamma_{p_j}^m} \mathbf{u}^* \mathbf{q}^{s_j}\, d\Gamma_{p_j} - \delta_j \boldsymbol{\Upsilon}_k^j F_{p_j} \right] \tag{2}$$

where $\Gamma_{p_j}^m$ is the pile-soil interface along the load-line *j* within the domain Ω_m; n_{ll}^m is the total number of load-lines in the domain Ω_m; $\mathbf{q}^{s_j}$ corresponds to the tractions along the pile-soil interface acting over the pile and within the soil; δ_j is equal to one if the load-line j contains the tip of a floating pile and zero otherwise; and $\boldsymbol{\Upsilon}_k^j$ is a three-component vector that represents the contribution of the axial force F_{p_j} at the tip of the j^{th} load-line.

The boundaries Γ^m are discretized into quadratic elements of triangular and quadrilateral shapes with six and nine nodes, respectively. Once all boundaries have been discretized, eq. (2) can be written, for each region Ω_m, in all nodes on Γ^m in order to obtain a matrix equation of the type

$$\mathbf{H}^{ss}\mathbf{u}^s - \mathbf{G}^{ss}\mathbf{p}^s - \sum_{j=1}^{n_{ll}^m} \mathbf{G}^{sp_j}\mathbf{q}^{s_j} + \sum_{j=1}^{n_{ll}^m} \delta_j \boldsymbol{\Upsilon}^{sj} F_{p_j} = 0 \tag{3}$$

where $\mathbf{u}^s$ and $\mathbf{p}^s$ are the vectors of nodal displacements and tractions of boundary elements; $\mathbf{H}^{ss}$ and $\mathbf{G}^{ss}$ are coefficient matrices obtained by numerical integration over the boundary elements of the fundamental solution times the corresponding shape functions; and $\mathbf{G}^{sp_j}$ is the coefficient matrix obtained by numerical integration over load-line j of the fundamental solution times the shape functions of the piles, when the unit load is applied on Γ^m.

Furthermore, eq. (2) will be also applied on internal nodes belonging to load-line $\Gamma^m_{p_i}$, so that one can write

$$\mathbf{c}\,\mathbf{u}^{p_i} + \mathbf{H}^{p_i s}\mathbf{u}^s - \mathbf{G}^{p_i s}\mathbf{p}^s - \sum_{j=1}^{n^m_{ll}} \mathbf{G}^{p_i p_j}\mathbf{q}^{s_j} + \sum_{j=1}^{n^m_{ll}} \delta_j \boldsymbol{\Upsilon}^{p_i j} F_{p_j} = 0 \tag{4}$$

where $\mathbf{H}^{p_i s}$ and $\mathbf{G}^{p_i s}$ are coefficient matrices obtained by numerical integration over the boundary elements of the fundamental solution times the corresponding shape functions; and $\mathbf{G}^{p_i p_j}$ is the coefficient matrix obtained by numerical integration over load-line j of the fundamental solution times the shape functions of the piles, when the unit load is applied on load-line $\Gamma^m_{p_i}$. Here, $\mathbf{u}^{p_i}$ is the vector of nodal displacements of the load-line *i*, which is multiplied by vector $\mathbf{c}$, valued $1/2$ in positions corresponding to pile nodes placed on a smooth surface (as e.g. pile heads) and the unity in the rest of positions. Note that a pile head node and a boundary node can coincide on the same point. When this happens, there exist two nodes with identical coordinates. Then, two equations, one written for the surface node and another written for the load-line node, will be equivalent, but free-terms will occupy different positions on the coefficients matrix, not yielding a singular system of equations.

On the other hand, piles are modelled by FEM as vertical beams according to the Euler-Bernoulli hypothesis, and are discretized using a three-node element with 13 degrees of freedom defined on it: one vertical and two lateral displacements on each node, and two rotations θ on each one of the extreme nodes, one about x_1 axis and another one about x_2. To do so, the time-harmonic elastic behaviour of the piles, considered as one-dimensional beams, is considered to be described by an equation of the type

$$\bar{\mathbf{K}}\,\mathbf{u}^p = \mathbf{F}^{ext} + \mathbf{Q}\,\mathbf{q}^p \tag{5}$$

where $\bar{\mathbf{K}} = \mathbf{K} - \omega^2\mathbf{M}$, being $\mathbf{M}$ and $\mathbf{K}$ the mass and stiffness matrices of the pile, ω the circular frequency of excitation, $\mathbf{u}^p$ the vector of nodal translation and rotation amplitudes along the pile, $\mathbf{F}^{ext}$ the vector of external forces over the pile, $\mathbf{q}^p$ the vector of tractions along the pile-soil interface, and $\mathbf{Q}$ the tractions-to-equivalent nodal forces matrix.

It is worth noting that, as it is assumed that the soil continuity is not altered by the presence of the pile, the value of distributed mass assigned to the pile should be modified as $\bar{m} = A(\rho_p - \rho_s)$ so as not to overestimate the total mass introduced in the model, being ρ_p and ρ_s the pile and soil densities.

Now, a global system of equations must be built using the expressions defined above. The links between piles and soil that will allow to do the coupling are the tractions $\mathbf{q}^{s_j} = -\mathbf{q}^{p_j}$ along the pile-soil interface and the displacements $\mathbf{u}^{p_j}$ along the pile *j*.

Eq. (5), for the pile *j*, becomes

$$\bar{\mathbf{K}}^{p_j}\,\mathbf{u}^{p_j} - \mathbf{F}_{p_j} + \mathbf{Q}\,\mathbf{q}^{s_j} = \mathbf{F}^j_{top} \tag{6}$$

Then, imposing equilibrium and compatibility conditions along the pile-soil interfaces, and assuming the tractions $\mathbf{q}^s$ as positive, eqs. (3), (4) and (6) can be rearranged in a system of equations representing the layered soil – pile foundation problem. For a uniform half-space, the system is of the form

$$\begin{bmatrix} \mathbf{H}^{ss} & -\mathbf{G}^{sp} & \boldsymbol{\Upsilon}^s & \emptyset \\ \mathbf{H}^{ps} & -\mathbf{G}^{pp} & \boldsymbol{\Upsilon}^p & \mathbf{C}' \\ \emptyset & \mathbf{Q} & \mathbf{I}' & \bar{\mathbf{K}} \end{bmatrix} \begin{bmatrix} \mathbf{u}^s \\ \mathbf{q}^s \\ \mathbf{F}_p \\ \mathbf{u}^p \end{bmatrix} = \mathcal{B} \tag{7}$$

being $\mathcal{B}$ the known right-hand vector when all external conditions have been imposed, and the vector of unknowns

$$\mathbf{x} = \{\mathbf{u}^s, \mathbf{q}^{s_1}, \mathbf{q}^{s_2}, \ldots, \mathbf{q}^{s_n}, F_{p_1}, F_{p_2}, \ldots, F_{p_n}, \mathbf{u}^{p_1}, \mathbf{u}^{p_2}, \ldots, \mathbf{u}^{p_n}\}^T \tag{8}$$

In case of multilayered domains, the structure of the system is the same, though equilibrium and compatibility fully bonded contact conditions have to be imposed over the different interfaces of the problem. Also note that piles in a group have been assumed to be fixedly connected to a rigid pile cap.

The dynamic behaviour of pile supported multistorey structures composed by any number of vertical extensible piers and horizontal rigid slabs (see fig. 2) is addressed in this work. Piers are modelled as massless Euler-Bernoulli beams, with axial and lateral deformation, and with hysteretic damping through a complex valued stiffness of the type $k = Re[k](1+2\mathrm{i}\zeta)$. Torsional stiffness is not considered in the piers. The principal axes of inertia of rigid slabs are assumed to be parallel to the global coordinate axes, though the position of their centre of gravity on the horizontal plane can change between storeys.

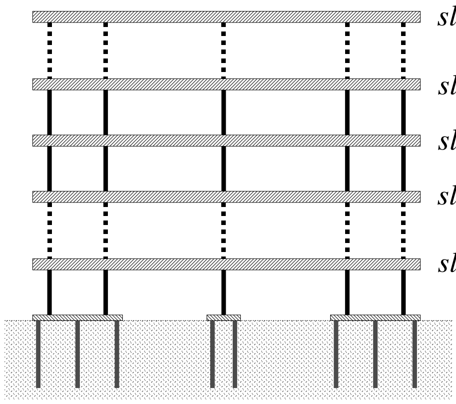

Figure 2: Two-dimensional sketch of considered pile supported structures

In order to write the equations directly in terms of slabs displacements and rotations (most interesting parameters in this kind of study), all DoF at piers ends are condensated to the centre of gravity of slabs and pile caps.

After defining a general element inter-storey stiffness matrix, the general assembly process of the Finite Element Method can be followed to build a discretized equation of motion for the structure of the form

$$\left(\mathcal{K} - \omega^2 \mathcal{M}\right) \mathcal{X} = \mathcal{F} \tag{9}$$

where $\mathcal{K}$ is the global stiffness matrix of the structure, $\mathcal{X}$ is the vector of displacements and rotations at slabs, $\mathcal{F}$ is the vector of external forces over the structure and $\mathcal{M}$ is the matrix of inertial properties of the structure, defined at each slab.

Finally, the way in which eqs. (5), (3), (4) and (9) are arranged into a global system of equations depends on the specific configuration and the boundary conditions, but equilibrium and compatibility fully-bonded contact conditions over the different interfaces of the problem are always imposed. The most general situation is that of a problem in which there exist multiple superstructures founded on different pile caps on a layered soil, being the system subject to external forces and/or incident seismic waves. In such a general case, the system of equations is of the form

$$\mathcal{A}\left\{\mathbf{u}^s, \mathbf{p}^s, \mathbf{q}^s, \mathbf{F}_p, \mathbf{u}^p, \mathbf{X}^j, \mathbf{F}_{top}, \mathbf{f}_o\right\}^T = \mathcal{B} \tag{10}$$

where $\mathcal{A}$, whose structure is sketched in fig. 3, is the square matrix of coefficients, and $\mathcal{B}$ is the known vector, both computed by rearranging the equations and prescribing the known boundary conditions. The vector of unknowns includes the displacements $\mathbf{u}^s$ and/or tractions $\mathbf{p}^s$ at boundary element nodes, the tractions at pile-soil interface $\mathbf{q}^s$, the forces at pile tips $\mathbf{F}_p$, the nodal translations and rotations on pile nodes $\mathbf{u}^p$, the degrees of freedom defined at the structures $\mathbf{X}^j$, the reactions at pile-cap joints $\mathbf{F}_{top}$ and the forces at structure base $\mathbf{f}_o$.

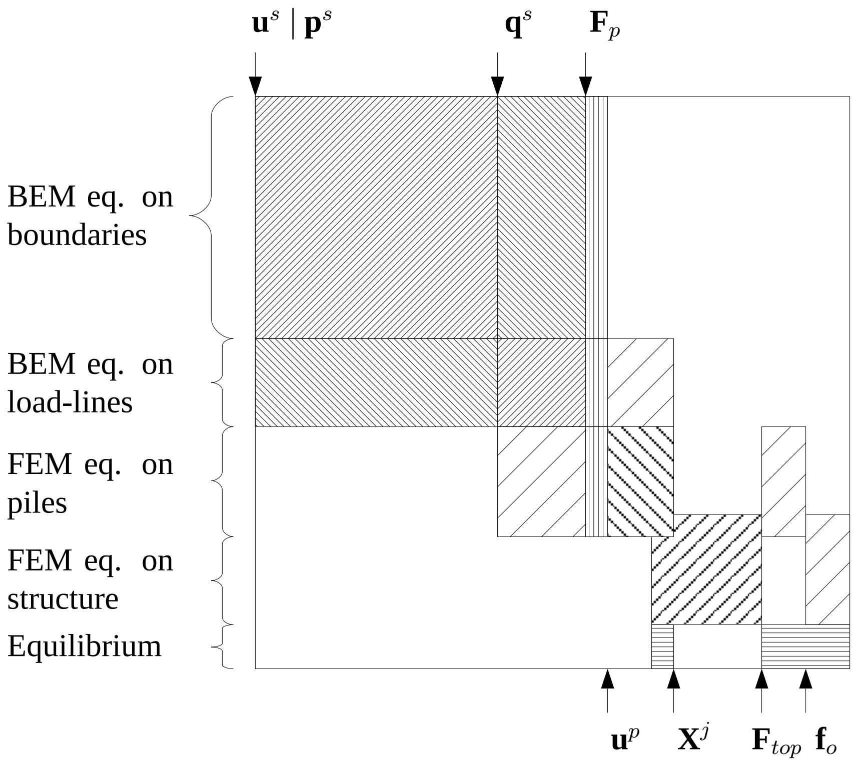

Figure 3: Structure of the system matrix of coefficients $\mathcal{A}$

3 NUMERICAL RESULTS

3.1 Definition of the problem

The system under investigation is composed of several neighbouring one-storey linear shear structures, three-dimensionally distributed, founded on 3×3 fixed-head pile groups embedded on a viscoelastic half-space. A plane sketch of the problem is depicted in fig. 4, where the geometric properties of buildings and piles are labelled. Pile groups are defined by length L and sectional diameter d of piles, centre-to-centre spacing between adjacent piles s and foundation halfwidth b, being in this specific case $b = s$. The rest of parameters are: centre-to-centre spacing between adjacent foundations D, fixed-base fundamental period T and structural damping ratio ζ, cap mass m_o and moment of inertia I_o, structure effective height h and structure effective mass m. Distance D between adjacent foundations is expressed as a fraction of the soil wave length at the soil-structure fundamental frequency $\lambda = c_s\tilde{T}$, being c_s the soil shear wave velocity.

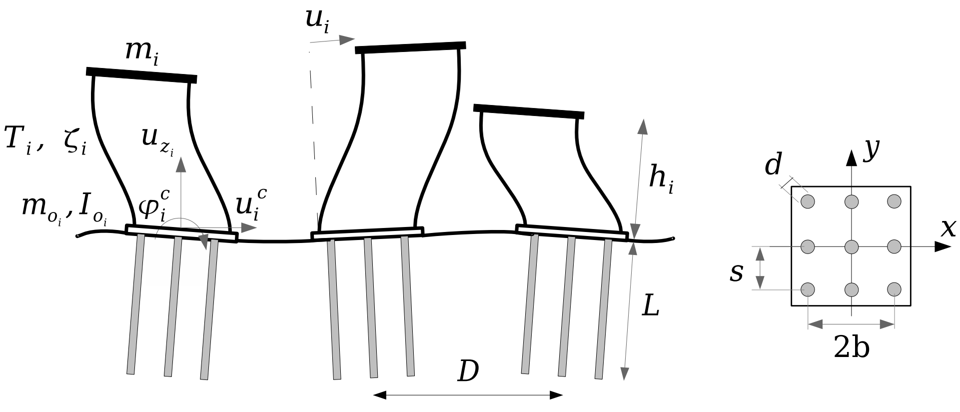

Figure 4: Geometric definition of the problem.

In this work, as a first approximation and also in order to focus on SSSI, superstructures are modelled as one-degree-of-freedom shear buildings in its fixed-base condition. However, these may represent either one-storey constructions or the fundamental mode of multi-mode structures. Subsequently, h, m and ζ must be generally understood as first-mode equivalent height, mass and damping ratio. On the other hand, note that fig. 4 is a two-dimensional representation of the three-dimensional model used herein. This way, eight degrees of freedom are considered on each foundation-superstructure subsystem: two lateral deformations of the structure u and two foundation translations u^c along axes x and y, one vertical displacement u_z, two rocking motions φ around horizontal axes and one rotational motion ϕ around the vertical axis. Note that vertical motions of cap and storey have been forced to be identical because buildings are modelled as purely shear structures.

The dynamic behaviour of several configurations under vertically incident plane S waves (producing motions on the y axis) or Rayleigh waves (moving along the y axis from $y < 0$ to $y > 0$), is analysed. To this end, the response of each structure in the group is compared to that of the single-structure-soil system in order to find out whether or not structure-soil-structure interaction effects between two or more buildings can be of importance. Note that in all configurations the distance D between adjacent structures is measured in parallel to x and y axes, and is the same between all structures in the same problem.

The mechanical and geometrical properties of pile foundations and soil are defined by the following parameters: piles separation ratio $s/d = 5$, pile-soil modulus ratios $E_p/E_s = 100$ and 1000, soil-pile density ratio $\rho_s/\rho_p = 0.7$, piles aspect ratio $L/d = 15$, soil damping coefficient $\beta = 0.05$ and Poisson ratio $\nu_s = 0.4$.

On the other hand, the most important parameters to define the superstructure dynamic behaviour are: structural aspect ratios $h/b = 2$, 3 and 4; structure-soil stiffness ratio $h/(T\,c_s) = 0.3$; and structural damping ratio $\zeta = 0.05$. Other parameters are: foundation mass moment of inertia $I_o = 5\%$, 2.2% and 1.25% of mh^2 for $h/b = 2$, 3 and 4, respectively; structure-soil mass ratio $m/4\rho_s b^2 h = 0.20$; and foundation-structure mass ratio $m_o/m = 0.25$. The values chosen for these three last parameters are considered to be representative for typical constructions, and similar values have been used by other authors before [5,6,9]. In any case, SSI and SSSI results are not significantly sensitive to its variation.

3.2 Steady-state response

The response of any of the structures is measured by its spectral lateral deformation, defined as $\delta u = \text{Abs}[\Omega^2 u/\omega^2 u_{ff}]$, where Ω is the fundamental frequency of the fixed-base structure, ω is the excitation frequency and u_{ff} is the horizontal free-field motion at the ground surface. The product of this value with the structural mass and the corresponding free-field horizontal acceleration at ground surface level yields the amplitude of the shear force at the base of the structure. The results shown in this section are plotted in terms of the amplification factor $\delta u/\delta u_1$, being δu the lateral deformation of any of the structures in the group and δu_1 the lateral deformation of a single building. Therefore, $\delta u/\delta u_1 > 1$ means that the presence of neighbouring structures amplifies the response of the building at a certain frequency, while $\delta u/\delta u_1 < 1$ would imply a beneficial effect of the grouping of the structures. All figures are plotted against the dimensionless frequency $a_o = \omega d/c_s$.

Fig. 5 presents the dynamic response, in terms of amplification of the spectral lateral deformation with respect to a single structure, of three identical buildings under vertically incident S waves, for $E_p/E_s = 1000$. Three different structural aspect ratios (h/b = 2, 3 and 4) are considered, being the fundamental frequency of a single structure in such soil $\tilde{a}_o \simeq 0.155$, 0.105 and 0.075 respectively. Three different distances between adjacent buildings ($D = \lambda/2$, $3\lambda/4$ and $\lambda/4$) have been studied. Shaking direction is assumed to be either parallel or perpendicular to the direction of alignment of the structures. It can be seen that the lateral response of a structure may vary significantly due to the presence of neighbouring buildings, in such a way that the lateral shear force at the base of the structure can be considerably amplified for frequencies around the fundamental frequency of the soil-structure system. The influence of SSSI varies from one position to another, as well as for different distances between structures and for different aspect ratios, and the response may even increase or decrease depending on the configuration. Amplifications of the order of $\pm 50\%$ can be achieved, but it appears that the central construction is usually subject to the strongest amplifications. It is also worth noting that even though problems with different h/b and the same D are not dimensionally equivalent, the same trends can be observed when D remains constant in terms of λ.

In the next two figures, groups of similar $h/b = 4$ structures under vertically incident S waves for $E_p/E_s = 100$ are studied, and two different distances between adjacent buildings ($D = \lambda/2$ and $D = \lambda/4$) are considered. Fig. 6 shows the response of a group of nine buildings, while fig. 7 shows results for a group of five aligned structures. It can be seen that, in general, the $D = \lambda/2$ configuration is much more unfavourable than the $D = \lambda/4$ situation, giving amplifications of the order of 150% and 100% for the central building, for the first and second cases respectively.

The response of these systems to incident Rayleigh waves is studied next. Figs. 8 and 9 present the dynamic response of groups of three similar $h/b = 2$ and $h/b = 4$ buildings, respectively, for $D = \lambda/2$. Results for $E_p/E_s = 1000$ and 100 are presented for Rayleigh waves impinging parallel or perpendicularly to the direction of alignment of the structures. The significant reduction in the spectral lateral deformation experienced by of the last structure to be hit by the waves denotes the important shielding effect produced by the presence of the other structures. Also, the amplifications, not larger than 25%, are smaller than those observed for vertically incident S waves, and are even less important for increasing structural aspect ratios.

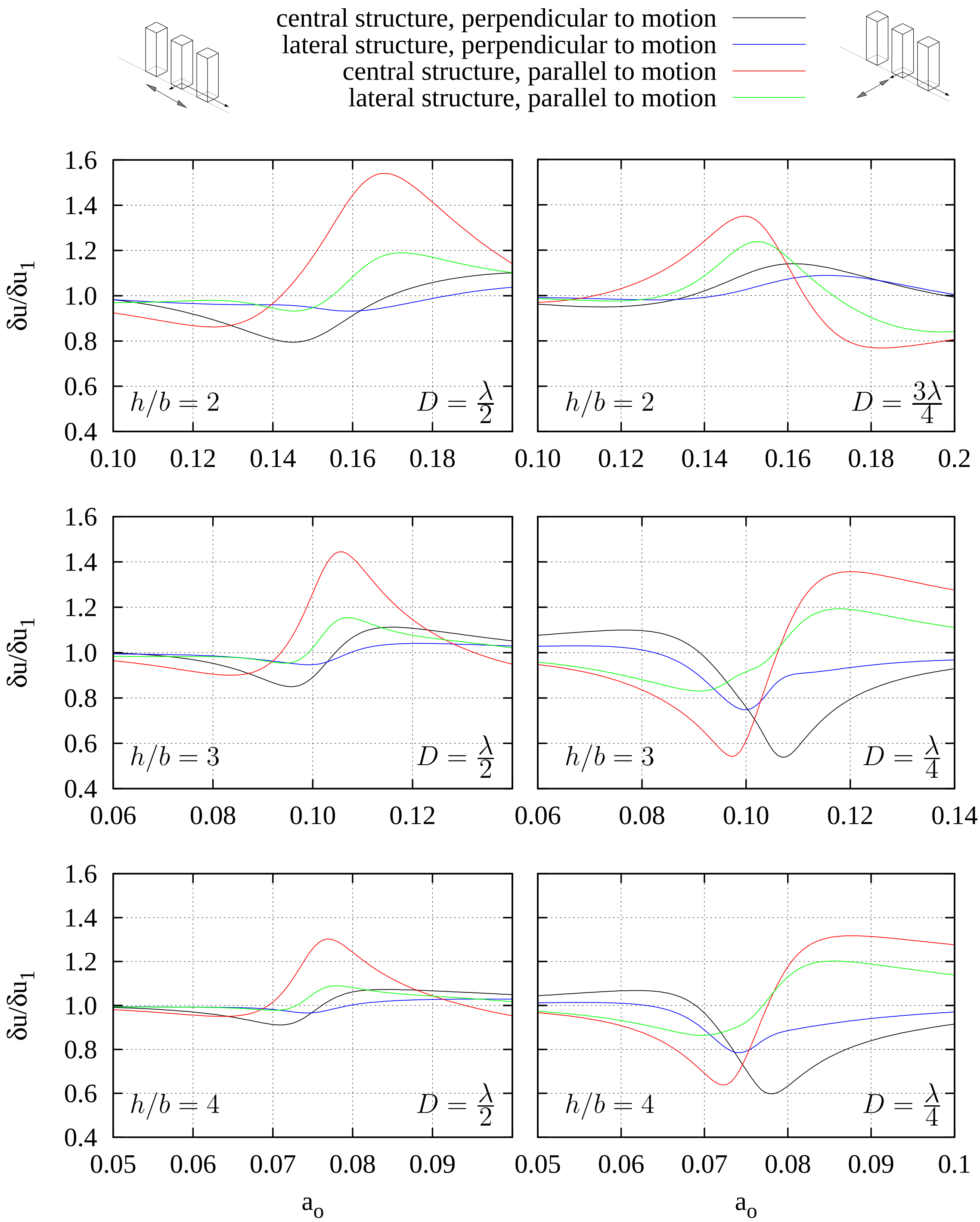

Figure 5: Amplification factors for the spectral lateral deformation due to the interaction among three structures of identical fundamental frequencies for different configurations under S waves. $E_p/E_s = 1000$

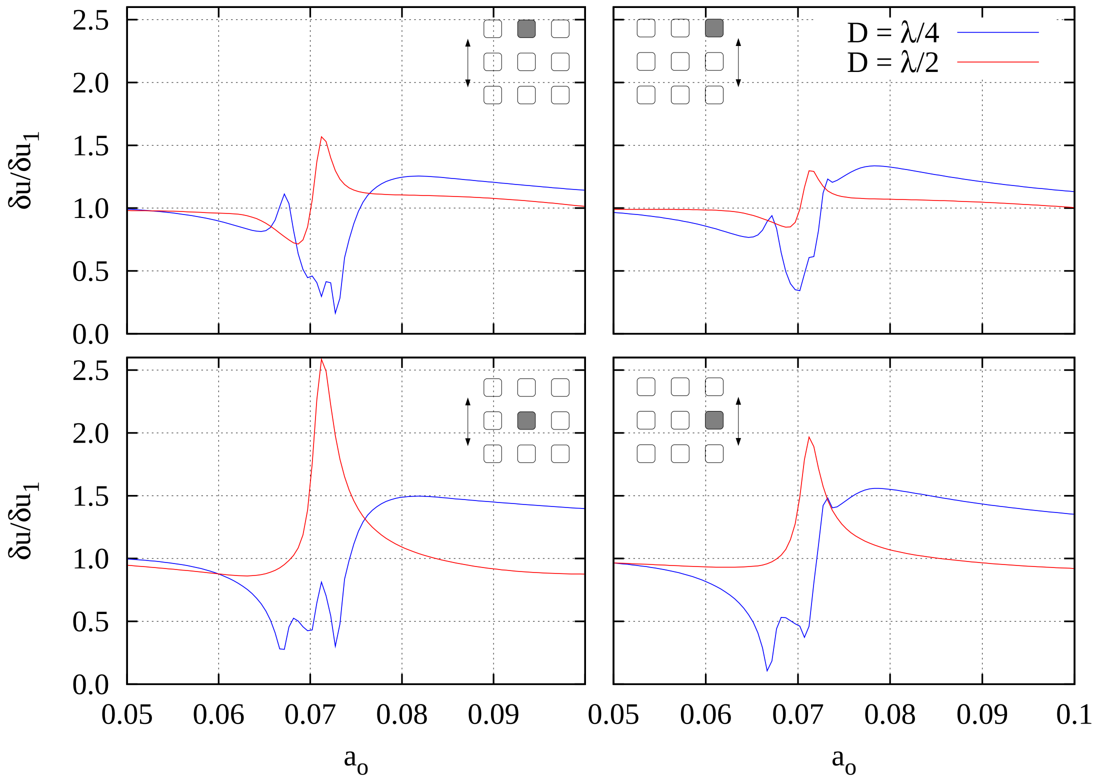

Figure 6: Amplification factors for the spectral lateral deformation due to the interaction among nine structures of identical fundamental frequencies for different configurations under S waves. $h/b = 4$. $E_p/E_s = 100$.

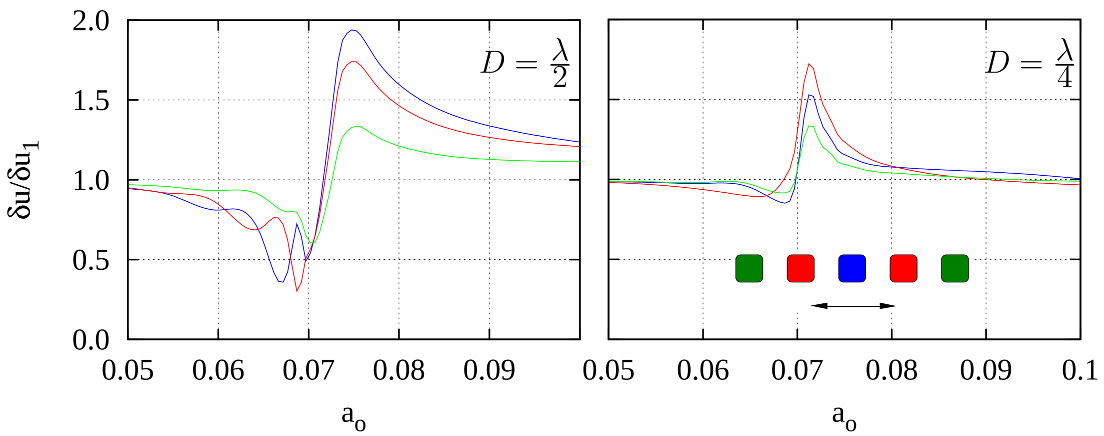

Figure 7: Amplification factors for the spectral lateral deformation due to the interaction among five structures of identical fundamental frequencies for different configurations under S waves. $h/b = 4$. $E_p/E_s = 100$.

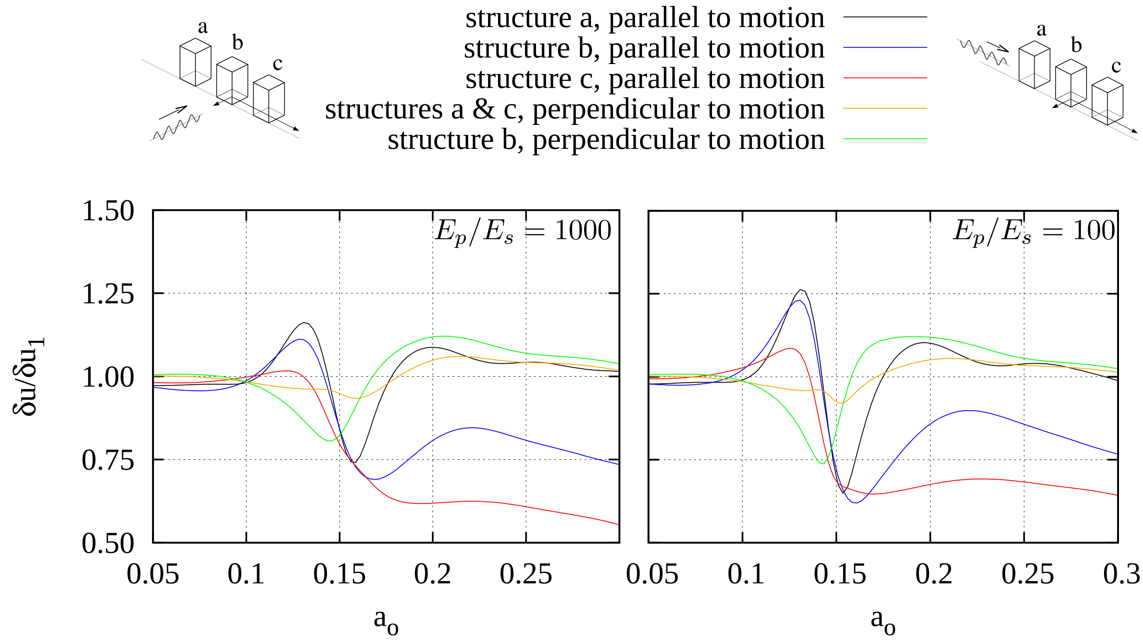

Figure 8: Amplification factors for the spectral lateral deformation due to the interaction among three structures of identical fundamental frequencies for different configurations under Rayleigh waves. $h/b = 2$. $D = \lambda/2$.

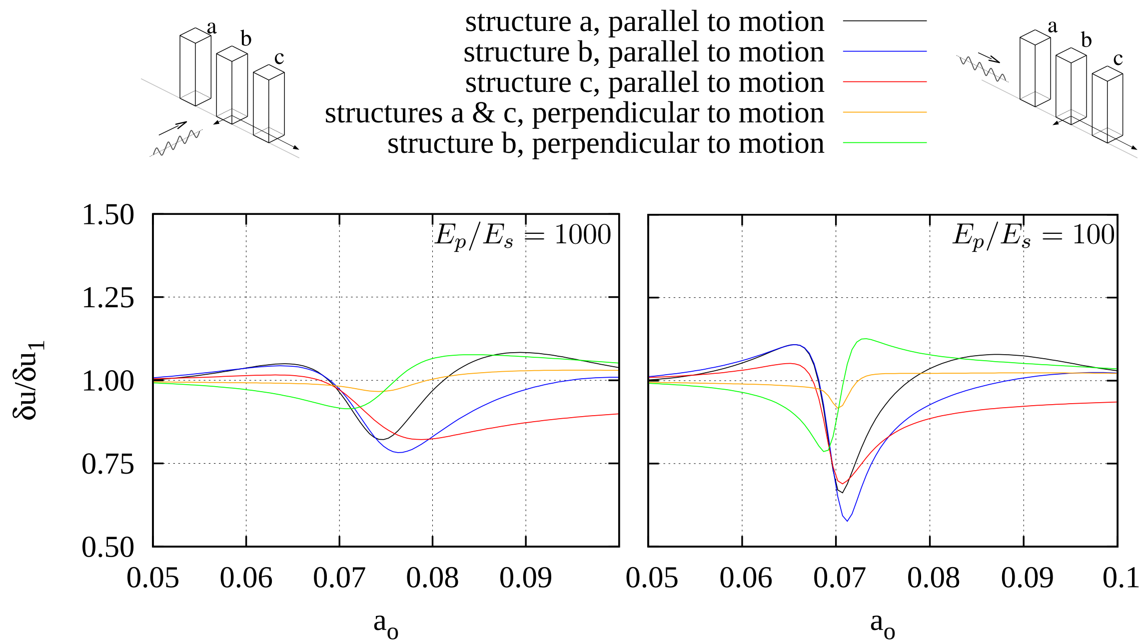

Figure 9: Amplification factors for the spectral lateral deformation due to the interaction among three structures of identical fundamental frequencies for different configurations under Rayleigh waves. $h/b = 4$. $D = \lambda/2$.

3.3 Earthquake response

After computing the corresponding transfer functions, acceleration time histories can be obtained for particular cases making use of the fast Fourier transform (FFT) algorithm. This way, selected accelerograms are presented in this section in order to measure the influence of SSSI on the seismic response of structures. The system is subjected to the N-S component of the Imperial Valley earthquake of May 18, 1940, recorded at the Imperial Valley Irrigation District substation in El Centro, California. Properties of soil and piles used to compute these results, only as an example, are summarised in table 1, being $h/b = 4$. It is worth saying that the soil-structure system fundamental period is $\tilde{T} \simeq 0.40\ s$.

Soil	**Piles**	**Structures**
$c_s = 239\ m/s$	$E_p = 2.76 \cdot 10^{10}\ N/m^2$	$T = 0.28\ s$
$\rho_s = 1750\ kg/m^3$	$\rho_p = 2500\ kg/m^3$	$m = 7 \cdot 10^5\ kg$
$\nu_s = 0.4$	$d = 1\ m$	$h = 20\ m$
$\zeta_s = 0.05$	$L = 15\ m$	$\zeta = 0.05$

Table 1: Soil, piles and structures properties.

Figs. 10 and 11 present results for groups of three and nine buildings, respectively, arranged as explained in the previous sections and with $D = \lambda/4$, while figs. 12 and 13 show the same results for $D = \lambda/2$. The accelerograms are computed at the building slabs. The free field response and the response of a single building are shown in all figures together with those of the central and the lateral or corner structure, depending on the case.

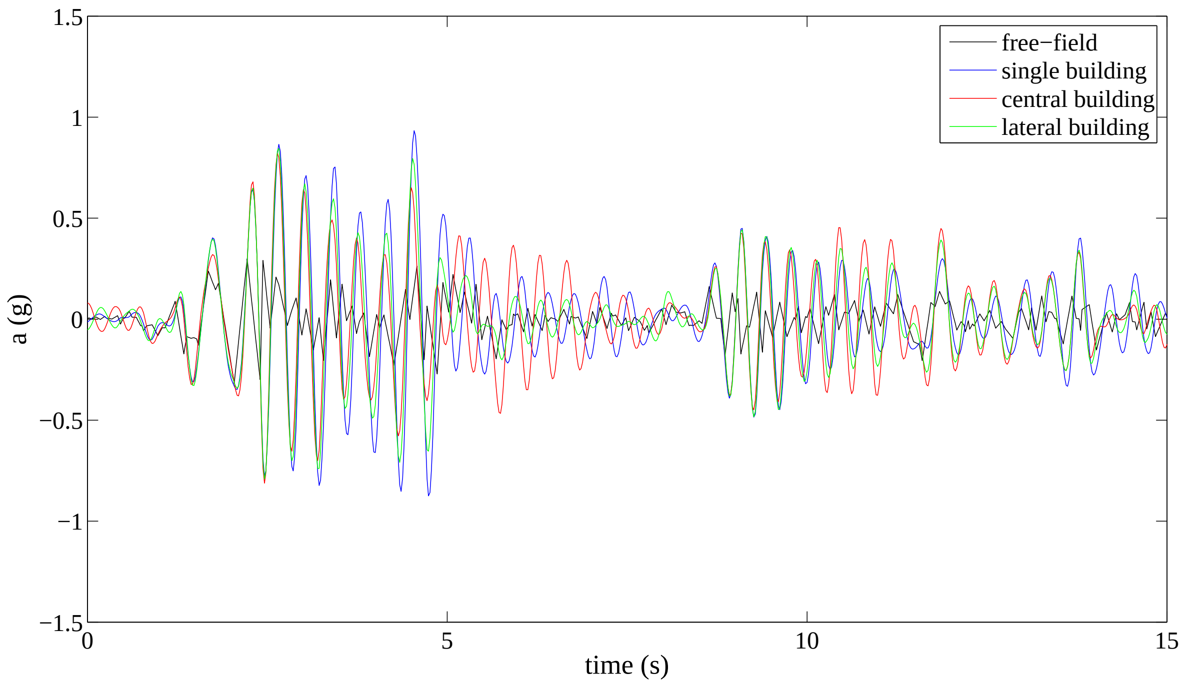

Figure 10: Acceleration time histories. Three buildings. $D = \lambda/4$

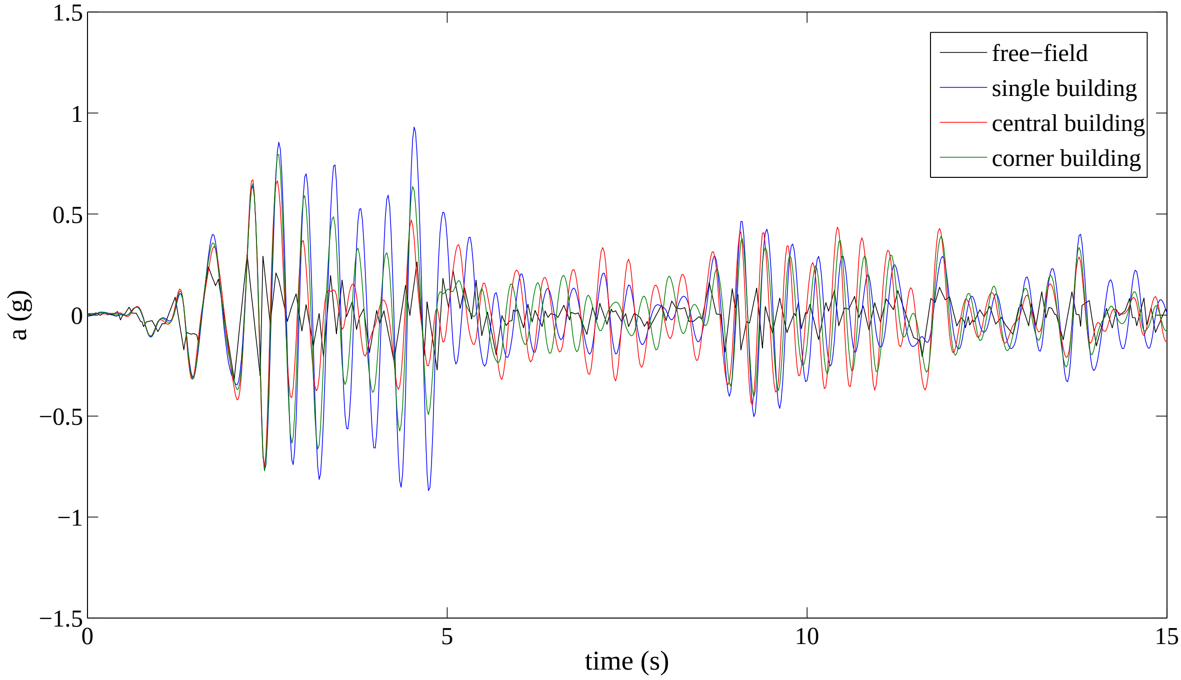

Figure 11: Acceleration time histories. Nine buildings. $D = \lambda/4$

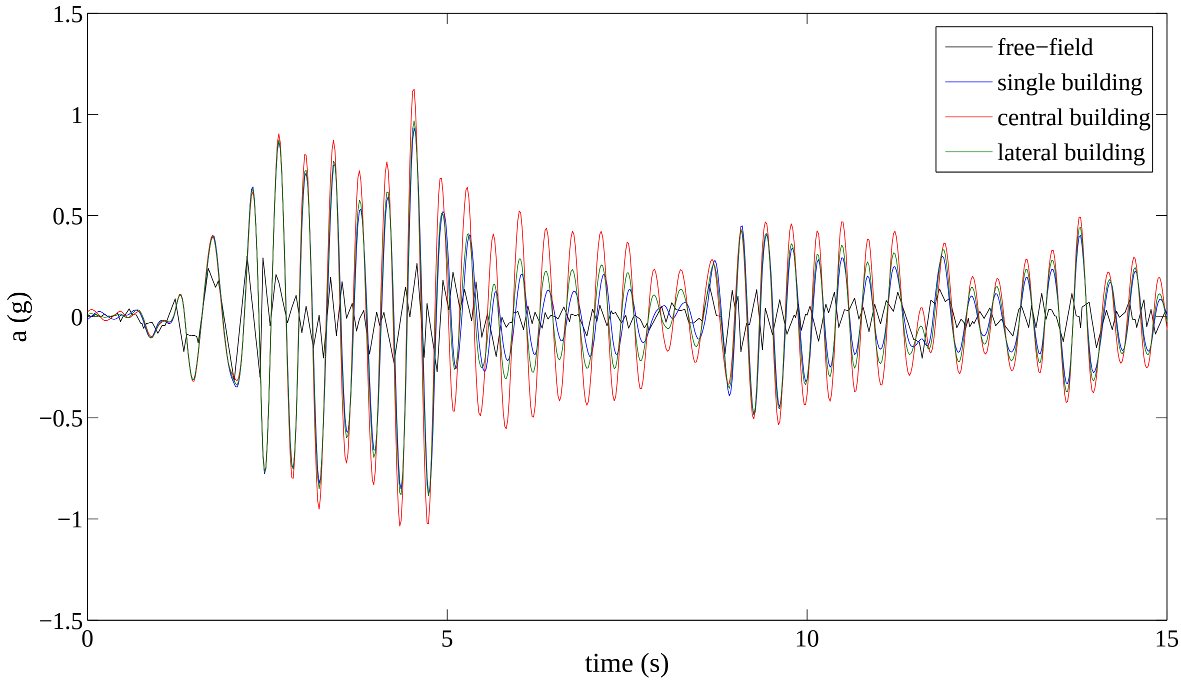

Figure 12: Acceleration time histories. Three buildings. $D = \lambda/2$

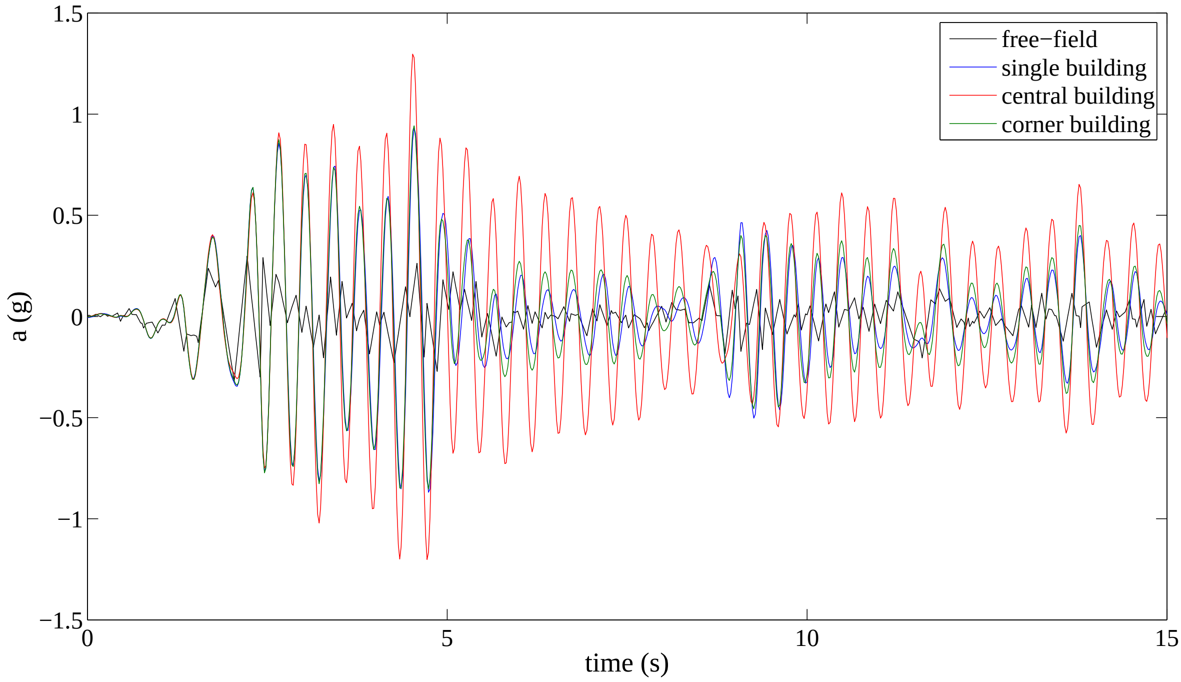

Figure 13: Acceleration time histories. Nine buildings. $D = \lambda/2$

It can be seen that, for $D = \lambda/4$, the response of the grouped structures tends to be smaller than that corresponding to a single building. On the contrary, for $D = \lambda/2$, the response is significantly amplified, mainly in the case of the structures occupying central positions.

4 SUMMARY AND CONCLUSIONS

- A 3D numerical procedure for the dynamic analysis of pile supported linear structures has been used to address the problem of through-soil interaction between neighbouring one-storey shear buildings.

- One-storey shear buildings, founded on 3×3 pile groups in a viscoelastic half-space, with different aspect ratios and separations between adjacent structures, were considered.

- SSSI effects have been found to be of importance in the case of groups of structures with similar dynamic characteristics, mainly in the structural response around the overall system fundamental frequency.

- Depending on the distance between adjacent buildings, the seismic response of each member of the group can be amplified or reduced.

- For vertically incident S waves, and for the set of properties and configurations selected for this work, the most unfavourable distance appears to be $D = \lambda/2$. For this separation between adjacent buildings, large amplifications have been observed in the response of groups of three and five aligned structures, and even larger motions for a square group of nine similar constructions. The highest amplifications occur at central constructions and when the impinging waves produce motions in the direction of alignment of the structures.

- When Rayleigh waves impinge in the same direction of alignment of the structures, the first building to be hit suffers large amplifications and, at the same time, shielding effects become apparent. The amplifications are smaller than those measured for vertically incident S waves
- The analysis of the time-history response of the grouped structures show that the magnitude of the seismic response can be significantly amplified, but it can also be reduced depending on the distance between adjacent structures, which could be used in the design of groups of buildings as a safety measure to reduce the seismic risk.
- Further studies about structure-soil-structure interaction phenomena and their influence on structural seismic risk are needed, as it has been shown that nearby buildings can significantly increase the seismic response of a structure.

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