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DIRECT APPROACH TO SEISMIC SOIL – STRUCTURE–INTERACTION ANALYSIS – BUILDING GROUP CASE

Abdelhacine Gouasmia[1], Kamel Djeghaba[2]

[1, 2]Department of Civil Engineering, University of Badji Mokhtar, BP 12 El Hadjar, 23220 Annaba, Algeria
E-mail: [1]Gouasmia.Abdelhacine@univ-annaba.org

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Abstract. Nowadays, the new and emerging concept of seismic structural design, the so-called performance-based design, requires careful consideration of all aspects involved in structural analysis. One of the most important aspects of structural analysis is soil-structure interaction (SSI). Such interaction may alter the dynamic characteristics of structures and consequently may be beneficial or detrimental to the performance of structures. In order to observe such effects we study the seismic response of an idealized small city composed of five equally spaced, five storey reinforced concrete buildings anchored in a soft soil layer overlaid by a rock half-space. Our results show predict response amplification of the buildings in the near field in accordance with the results observed in similar cases.

Keywords: soil-structure interaction, wave based methods, site effect, seismic response, response amplification.

1. Introduction

Nowadays, the new and emerging concept of seismic structural design, the so-called performance-based design, requires careful consideration of all aspects involved in structural analysis. One of the most important aspects of structural analysis is soil-structure interaction (SSI). Such interaction may alter the dynamic characteristics of structures and consequently may be beneficial or detrimental to the performance of structures (Gouasmia, Djeghaba 2007).

Soil conditions at a given site may amplify the response of a structure on a soil deposit. Not taking into account these structural response amplifications may lead to an under-designed structure resulting in a premature collapse during an earthquake.

Analytical methods of SSI concentrates mainly on single degree of freedom systems and analysis/design of long and important structures such as large bridges and nuclear power plants, and rarely on regular type buildings.

The main idea behind this investigation is motivated by the fact that there is still great uncertainty as to the significance of seismic soil-structure interaction (SSI) for ordinary structures typically encountered in Algeria (Gouasmia, Djeghaba 2005). There may be both beneficial and adverse effects of SSI. However, in many cases, SSI is simply ignored in design without establishing whether it will increase or decrease the response of the structure (Gouasmia, Djeghaba 2006). A second objective is that the probability of an earthquake of magnitude 7 or larger may occur in regions that have experienced strong earthquakes such as Chlef or Zemmouri (Algeria). Therefore, studies which include SSI effects will help to better predict the performance of structures during future earthquakes (The Boumerdes … 2003).

State of the art knowledge and analytical approaches require, that, the structure-foundation system to be represented by mathematical models that include the influence of the sub-foundation media. Analytical models were developed by finite element for numerical analysis.

Therefore, different analyses were performed on a simulated city made of a group of building composed of five storey reinforced concrete buildings. In fact,

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structure types of 1, 5, 10 and 20 storey buildings are typically encountered in Algeria. Such structures are generally designed following Algerian code requirements (RPA 2003) neglecting SSI effects (Règles … 2003). The objective of this study as reported herein focuses mainly on the numerical modelling of cities represented by structural groups of typical five storey reinforced concrete buildings incorporating special soft soil conditions, in order to assess the effects of SSI on the dynamic response of such structures (Gueguen 2000; Gupta, Trifunac 1991; Safak 1999; Veletsos, Meek 1974; Veletsos, Nair 1975; Avilés, Pérez-Rocha 1998).

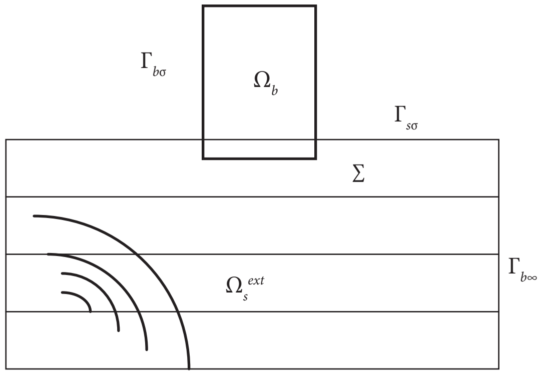

Fig. 1. Geometry and notations of the subdomains

2. Soil-structure interaction problem formulation

There are different methods to analyze SSI systems, they are generally subdivided in two main groups namely: direct and substructure methods. In the direct method part of the soil around the structure is modeled with special absorbent boundaries. This method is well suited for nonlinear material laws of the soil to be taken into account. In the substructure method, the soil-structure-interaction system is divided into two substructures, the soil medium and the structures. A result of this assumption is that only linear systems are handled (Baba *et al.* 1996).

A significant part of this study addresses the aspect of SSI effects. Conventional analysis applies the seismic excitation at the base of the structure, but current understanding suggests that this may not be accurate in cases where the structure rests on a compressible soil or where the properties of the foundation may alter the response of the structure. An accurate approach would be to analyze the entire SSI system as a whole, which includes modeling the structure, foundation, and the surrounding soil, and then calculating the response of the entire system (Kramer 1996).

Note that this approach is different from the one used in other research works (Safak 1999; Wolf 1994; Hughes 1987), wherein the foundation-building system is considered either to be shear wall with rigid or flexible foundations, or inverted pendulums attached to (mass-spring-dashpot systems).

Foundation wave induced vibrations are caused by earthquakes that pass through the soil (Fig. 1). A dynamic excitation is generated due to the interaction between the foundations and the soil, which requires the solution of a dynamic soil-structure interaction problem at the interface Σ between the foundation and the soil. Waves generated in the far field in the soil domain Ω_s^{ext} and impinge on the foundation of the structure Ω_b, which leads to a dynamic soil-structure interaction (DSSI) problem at the interface Σ between the soil and the structure.

The incident wave field interacts with the structure and generates vibrations. The foundation and the structure are coupled through the soil. First, the soil model is used to predict the incident wave field u_{inc} due to the passage of waves, accounting for dynamic foundation-soil interaction (Fig. 1). The incident wave field u_{inc} is defined on the semi-infinite layered soil domain $\Omega_s = \Omega_s^{ext} \cup \Omega_s^{int}$ without excavation of the interior soil domain Ω_s^{int}. Dynamic foundation-soil interaction is accounted for by means of the direct formulation (Burton, Miller 1971). The continuity of displacements is accounted for along the foundation-soil interface Σ_{rs}.

The next step is the propagation of the incident wave field to the structure Ω_b and the response is computed, accounting for dynamic soil-structure interaction (DSSI).

2.1. Governing equations of motions for the displacement in the structure

At first, the structure Ω_b is considered (Fig. 1). The boundary $\Gamma_b = \Gamma_{b\sigma} \cup \Sigma$ of the structure Ω_b is decomposed into a boundary $\Gamma_{b\sigma}$ where tractions $\overline{t_b}$ are imposed at the soil-structure interface Σ.

The displacement vector u_b of the structure satisfies the following Navier equation and boundary conditions:

$$\mathrm{div}\sigma_b(u_b) + \rho_b b = -\rho_b \omega^2 u_b \text{ in } \Omega_b, \quad (1)$$

$$t_b(u_b) = \overline{t}_b \text{ on } \Gamma_{b\sigma}, \quad (2)$$

$$u_b = u_s \text{ on } \Sigma, \quad (3)$$

$$t_b(u_b) + t_s(u_s) = 0 \text{ on } \Sigma, \quad (4)$$

where $\rho_b b$ is the body force on the structure and $t(u) = \sigma(u) \bullet n$ is the traction vector on a boundary with a unit outward normal vector n.

2.2. Formulation of the equations for the displacements in the soil

The displacement vector u_s of the soil satisfies Navier equation and the following boundary conditions:

$$\mathrm{div}\sigma_s(u_s)+\rho_s b=-\rho_s\omega^2 u_s \text{ in } \Omega_s^{ext}, \quad (5)$$

$$t_s(u_s)=0 \text{ on } \Gamma_{s\sigma}, \quad (6)$$

$$u_s=u_b \text{ on } \Sigma, \quad (7)$$

$$t_b(u_b)+t_s(u_s)=0 \text{ on } \Sigma, \quad (8)$$

where it is assumed in equation (5) that the body force $\rho_b b$ represents a constant that is directly applied on the semi-finite layered half-space Ω_s^{ext} and causes an incident wave field in the soil (Fig. 1).

The displacement vector u_s in the soil is generally decoupled using a Helmholtz decomposition, which results in a set of uncoupled partial differential equations representing the longitudinal and shear wave propagation.

2.3. Variational formulation

In this section, the equation of motion of the DSSI problem is approached using a variational form (Hughes 1987). The principle of virtual work states that the equilibrium of the structure requires that for any virtual displacement field δv imposed on the structure, the sum of the virtual work of the internal and the inertial forces is equal to the total virtual work of the external loads, which results in the following weak form integral equation:

$$\int_{\Omega_b}\varepsilon(\delta v):\sigma_b(u_b)d\Omega-\omega^2\int_{\Omega_b}\delta v.\rho_b u_b d\Omega=$$
$$\int_{\Omega_b}\delta v.\rho_b b d\Omega+\int_{\Omega_b}\delta v.\overline{t}_b d\Gamma+\int_{\Omega_b}\delta v.t_b(u_b)d\Sigma. \quad (9)$$

The volume integrals over Ω_b will result in the mass and the stiffness matrix of the structure. As the structure Ω_b has a finite dimension, the mass and the stiffness matrix can be calculated using the FEM. FEM procedures are widely used in structural analysis and only the basic principles of the FEM are needed in the discretisation of the scalar equation (9). Over the boundary Σ, the tractions $\mathbf{t}_s(\mathbf{u}_{sc}(\mathbf{u}_b))$ and $\mathbf{t}_s(\mathbf{u}_{inc}+\mathbf{u}_{d0})$ given in the surface integral are computed using the FEM.

For any virtual displacement field $\delta\underline{v}$ the virtual work equation must hold, and then the equation (9) is equivalent to:

$$(K_b-\omega^2 M_b+K_s)\underline{u}_b=f_b. \quad (10)$$

The stiffness matrix $\mathbf{K}_b$ and the mass matrix $\mathbf{M}_b$ of the structure are given by:

$$K_b=\int_{\Omega_b}\Omega\, B_b^T D B_b d, \quad (11)$$

$$M_b=\int_{\Omega_b}N_b^T\rho_b N_b d\Omega. \quad (12)$$

FEM is used to calculate the stiffness matrix $\mathbf{K}_b$ and the mass matrix $\mathbf{M}_b$ of the structure.

The dynamic stiffness matrix $\mathbf{K}_s$ of the semi-finite layered half-space is given by:

$$K_s=\int_\Sigma N_b^T t_s(u_{sc}(N_b))d\Sigma. \quad (13)$$

The vector $\mathbf{f}_b$ due to the external forces on the structure is defined by:

$$f_b=\int_{\Omega_b}N_b^T\ \rho_b b d\Omega+\int_{\Gamma_{b\sigma}}N_b^T\ \overline{t}_b d\Gamma-$$
$$\int_\Sigma N_b^T t_s(u_{inc}+u_{d0})d\Sigma. \quad (14)$$

Again finite element approach is used for the calculation of the tractions $\mathbf{t}_s(\mathbf{u}_{sc}(\mathbf{N}_b))$ in the dynamic stiffness matrix $\mathbf{K}_s$ of the soil and $\mathbf{t}_s(\mathbf{u}_{inc}+\mathbf{u}_{d0})$ in the external force vector $\mathbf{f}_b$.

The solution of the elastodynamics problem on the exterior domain Ω_s^{ext} having an embedded region Ω_s^{int} of finite extent, using a discretisation form of a displacement equation, is not unique at the eigenfrequencies of the embedded interior domain Ω_s^{int} with Dirichlet boundary conditions along the soil-structure interface Σ and free boundary conditions along the free surface Γ_{s0} (Burton, Miller 1971; Chen *et al.* 2002; Rizzo *et al.* 1985) and absorbent boundary conditions at the vertical borders of the bounded soil domain. This numerical deficiency problem occurs in the high frequency range, and it depends on the geometry of the foundation and the stiffness of the excavated soil. Therefore, the problem of fictitious frequencies is not very stringent for applications in seismic engineering, where the excitation frequencies are low (typically between 0 and 10 Hz).

In the frequency domain, hysteretic damping can be introduced using the correspondence principle and by replacing the stiffness matrix $\mathbf{K}_b$ by a complex stiffness matrix $K_b(1+2\xi i)$ with ξ a damping ratio.

The introduction of hysteretic damping in the equilibrium equation (10) results in:

$$(K_b(1+2\xi i)-\omega^2 M_b+K_s)\underline{u}_b=f_b. \quad (15)$$

If viscous damping is introduced, the equilibrium equation (10) of the SSI problem becomes:

$$(K_b + i\omega C_b - \omega^2 M_b + K_s)\underline{u}_b = f_b. \tag{16}$$

In the present work, proportional damping corresponding to defining a damping ratio $\xi_m \left(m = 1,...,q\right)$ for each mode ψ_m of the structure is used. The advantage of using proportional damping will result in a decoupled equilibrium equation. The corresponding damping matrix C_b is then computed as follows:

$$C_b = M_b \underline{\Psi} \mathrm{diag}(2\xi_m \omega_m) \underline{\Psi}^T M_b, \tag{17}$$

where the vector $\underline{\Psi}$ represents the eigenmodes $\underline{\psi}_m$ at the corresponding eigenfrequencies ω_m of the structure. In this work, the same damping ratio is used for all modes. Rayleigh damping is defined as a special case of proportional damping in which the damping matrix C_b is equal to:

$$C_b = \alpha M_b + \beta K_b. \tag{18}$$

where the parameters α and β can be calculated by $\alpha = 2\xi\omega_m\omega_n / \left(\omega_m + \omega_n\right)$ and $\beta = 2\xi / \left(\omega_m + \omega_n\right)$, with ω_m and ω_n being two specific eigenfrequencies and ξ the damping ratio that applies to both frequencies.

3. Finite element analysis

We assume plane strain conditions, that is, all frames parallel to the plane of calculation in (Fig. 4) deform identically. This represents regularly spaced frames in the transverse direction.

3. 1. Soil elements

A triangular element having 15 nodes is chosen for the 2D analysis (Fig. 2). This element is very powerful and provides accurate results of stresses and strains. The stresses are evaluated at the stress points contained in the element as indicated in (Fig. 2).

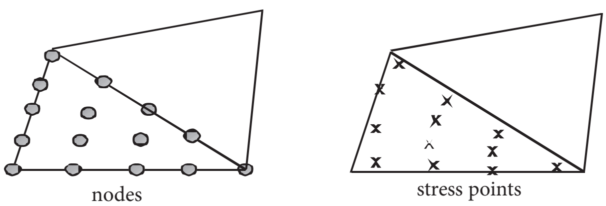

Fig. 2. Position of nodes and stress points in soil elements

As a first approximation we used Mohr-Coulomb model for soil behaviour. This model involves five parameters: Young's modulus E, Poisson's ratio ν, cohesion c, friction angle ϕ, and the dilatancy angle ψ.

3.2. Interfaces

Interfaces are used to model the soil structure interaction. A typical application of interfaces would be to model the interaction between the soil and the foundation. The interaction is modelled by choosing a suitable value for the strength reduction factor in the interface. This factor relates the interface strength (foundation friction and adhesion) to the soil strength (friction angle and cohesion).

3.3. Interface elements

Figure 3 shows how interface elements are connected to soil elements. When using 15-node soil elements, the corresponding interface elements are defined by five pairs of nodes. In the same figure, the interface elements are shown to have a finite thickness, but in the finite element formulation the coordinates of each node pair are identical, which means that the interface element has a zero thickness.

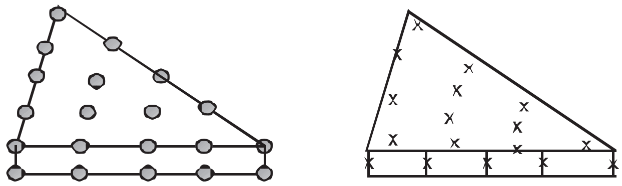

Fig. 3. Distribution of nodes and stress points in interface elements and connection with soil elements

3.4. Interface strength

Coulomb's criterion is used to model the elastic-plastic behaviour of interfaces, where both small and large displacements are taken into account. Allowing for proper modelling of soil-structure-interaction problems.

For small displacements the interface shear stress τ is given by:

$$\left|\tau\right| < \sigma_n \tan\varphi_i + c_i. \tag{19}$$

For plastic behaviour τ is given by:

$$\left|\tau\right| = \sigma_n \tan\varphi_i + c_i, \tag{20}$$

where ϕ_i and c_i are the friction angle and cohesion of the interface and σ_n and τ are the normal stress and shear stress at the interface element. The strength properties of interfaces are linked to the strength properties of the soil layer. The interface associated strength reduction factor (R_{inter}) is calculated from the soil properties by applying the following:

$$c_i = R_{inter} c_i, \tag{21}$$

$$\tan \varphi_i = R_{inter} \tan \varphi_{soil} \leq \tan \varphi_{soil}, \tag{22}$$

$$\psi_i = 0° \text{ for } R_{inter} < 1, \tag{23}$$

Otherwise $\psi_i = \psi_{soil}$.

3.5. Boundary conditions

The unbounded nature of the soil medium requires special Boundary Conditions (BC) that does not reflect seismic waves into the soil-structure domain. Various models of BC exist that enable the energy transmission (Lysmer, Kuhlmeyer 1969), the most commonly used in the FEM are of the viscous type. The local viscous boundaries should be placed far away from the structure in order to obtain realistic results. From recent studies it is recommended that the location of the transmitting boundary to be selected far away 8-10 times of the foundation base width (Rosset, Kausel 1976). The BC used in this study are based on the method described by Lysmer and Kuhlmeyer (Rosset, Kausel 1976). The normal and shear stresses absorbed by a viscous damper are:

$$\sigma_n = -c_1 \rho V_p \dot{u}_x, \tag{24}$$

$$\tau = -c_2 \rho V_s \dot{u}_y, \tag{25}$$

where, r is the density of the materials, V_p and V_s are the P wave velocity and the S wave velocity, respectively; c_1 and c_2 are special relaxation coefficients that are introduced to improve the absorption effect of the viscous damper. For practical applications, reasonable values are: $c_1 = 1$ and $c_2 = 0.2$. However, these values do not assure fully absorbed S waves, and additional research is needed on this point.

4. Numerical Application

The proposed analysis model is applied to study the dynamic responses of five 5 storey R/C buildings to earthquake excitation in the time domain. The computational model employed in this section is shown in (Fig. 4), where the numerical results are obtained using Plaxis programs. Two cases are considered, case 1 corresponds to one R/C building resting on the surface of a soft soil layer (Fig. 5). While, case 2 corresponds to five R/C buildings resting on the surfa×ce of a soft soil layer (Fig. 6). Points A, B, C, D and E indicating locations where displacements and accelerations are calculated.

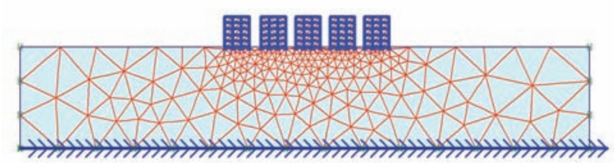

Fig. 4. Mesh of the soil-structure interaction

Automatic mesh generation of finite element meshes is used in this study. A special version of the triangle mesh generator is used (Ingenieurs …), which results in unstructured meshes. The numerical performance of such meshes is usually better than for structured regular meshes.

Geometric nonlinearity is important in our case which involves buckling of slender beams and soil mediums. Therefore an updated mesh analysis is used based on the updated Lagrangian formulation (Mcmeeking, Rice 1975).

The PARKFIELD (CALIFORNIA) earthquake accelerogram (Fig. 7) is employed as the horizontal ground motion applied to the analysis model. The input accelerogram is in the standard SMC format from the USGS office.

The five buildings are of the same type (5 storey). They are 3.0 m × 3 = 10.5 m wide and their total height

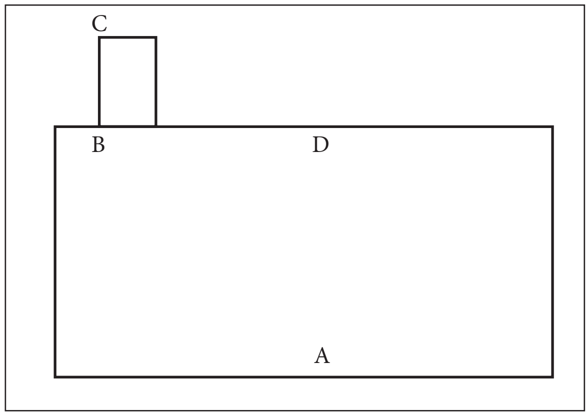

Fig. 5. Case 1, corresponds to one building-soil system

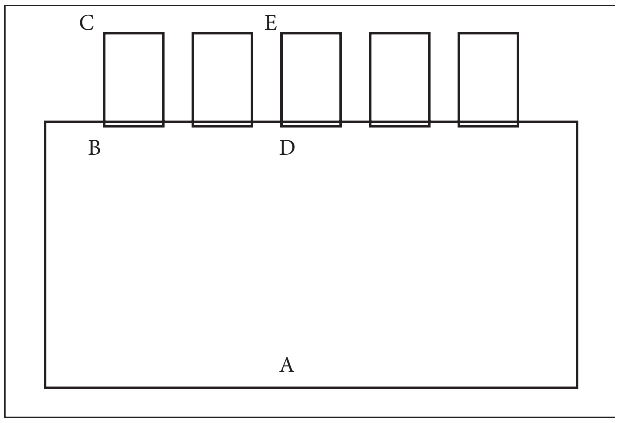

Fig. 6. Case 2, corresponds to five buildings-soil system

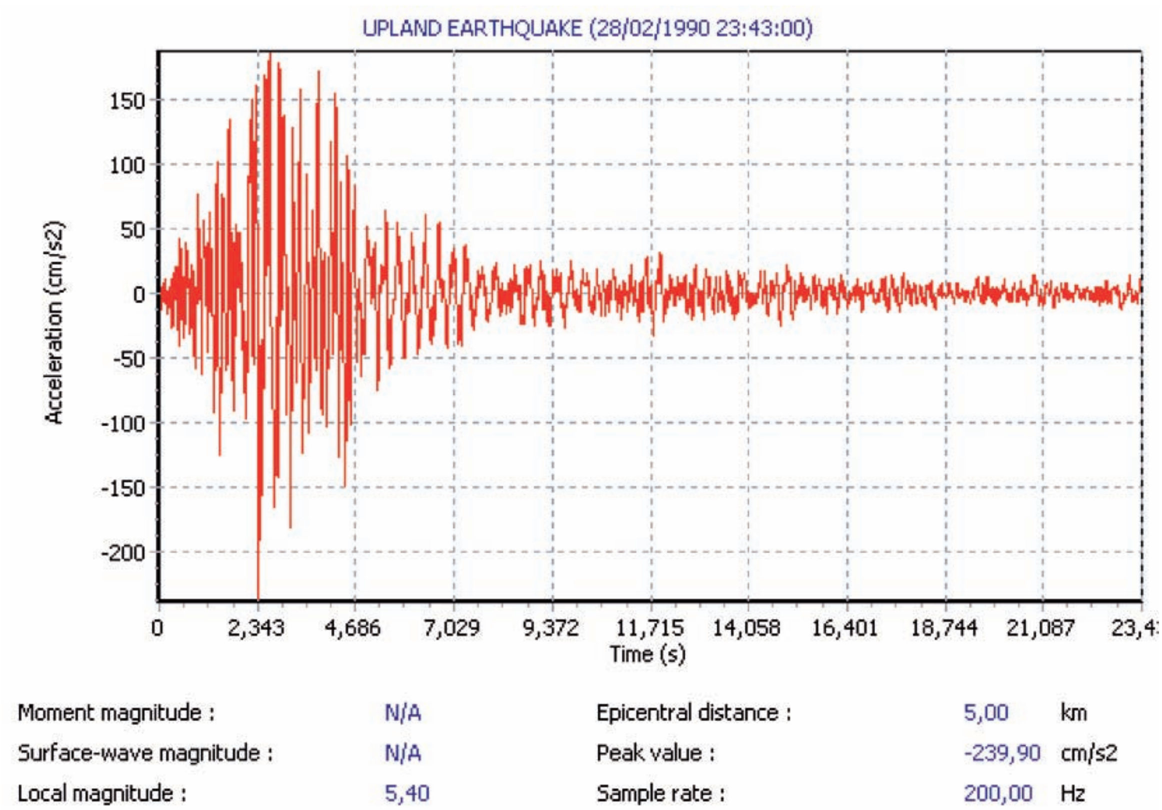

Fig. 7. PARKFIELD (CALIFORNIA) earthquake accelerogram

from the ground level is 4.08 m × 6 = 24.48 m. The dead load acting on each floor is up to 2.92 t/m and the live load up to 1.18 t/m.

The following material properties are used:

Concrete: Young's modulus E = 33300 x10^6 N/m^2, Poisson's ratio ν = 1/3 and Density ρ = 2500 kg/m^3 Mohr-Coulomb Soil: Young's modulus E = 4,532 x10^4 KN/m^2; Poisson's ratio ν = 0.2; cohesion c = 2KN/m; Friction angle � =24°; Shear wave velocity V_s = 107.50 m/s; Compressional wave velocity V_p = 175.60 and Density ρ = 1600 kg/m^3.

Soft soil layer depth = 30.0 m.

5. Discussion of results and conclusions

A numerical model for the prediction of wave induced vibrations in buildings has been developed and used for analysis. The coupled soil-structure system takes account of the free field wave induced vibrations in buildings; both models are based on a direct formulation approach for dynamic SSI problems.

A study on the determining factors for wave induced vibrations in buildings has been performed; the response of the building has been calculated for a one building type case and five buildings type case. The importance of SSI for the two cases in the dynamic SSI problem has been investigated. The soft soil condition has been adopted. The conclusions from the investigation of the modal characteristics of the structure and the response in terms of displacement and acceleration in different points of the SSI system are summarized as follows:

1. The stiffness of the soil plays an important role in the free field response. Higher vibration amplifications occur in the case of a soft soil.
2. The building displacements become large, but the motion dies away quickly in the building. There is an indication of rather large response not only in the buildings, but also on the ground level, and in the layer. This was also discussed by some authors for a periodic distribution of identical blocks (Clouteau, Aubry 2001).
3. The buildings constitute diffractors whereby seismic surface waves are locally generated, which then travel back and forth in between pairs of buildings, thus resulting in the coupling of the motions of the buildings via the soil so as the result will be a longer duration of the shaking inside the buildings which is longer than the one observed in the one-building case (this is not shown on the figures).

4. The time histories represented in Figs. 8–17, call for the following comments.
5. In Figs. 8–17, it can be observed that new effects, related to duration lengthening and beating, make their appearance in the response of the 5-buildings case (Fig. 17). Collective causes such as interference and building-soil-building interaction dominate during the first phase of shaking and radiation damping dominates the response during the later phase.
6. The peak amplitude of building response is larger at locations of the 5-buildings case than in the 1-building case (Figs 11, 14), and the longer duration of response in certain blocks of the 15-building case makes these buildings more vulnerable than the isolated building.
7. The cumulative response at the top of the buildings varies significantly from one building to another, corresponding to increased vulnerability for the 5-buildings case (case 2), which suggests that some of the buildings may suffer severe damage, while others will go unaffected, as a result of an earthquake in a city such as this one.

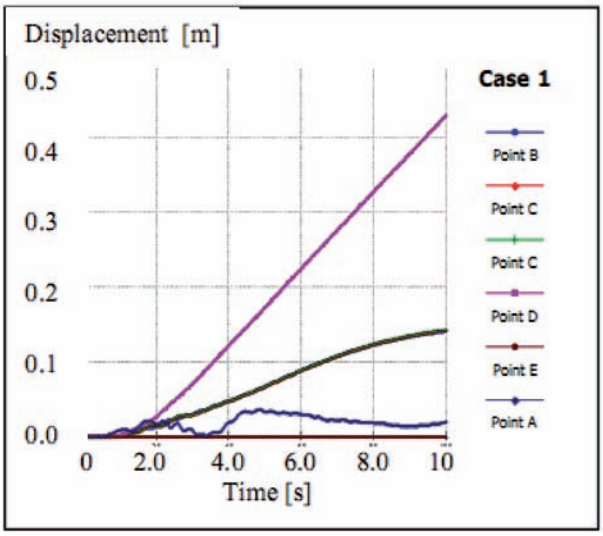

Fig. 8. Case 1, displacement time history in different points of the SSI system

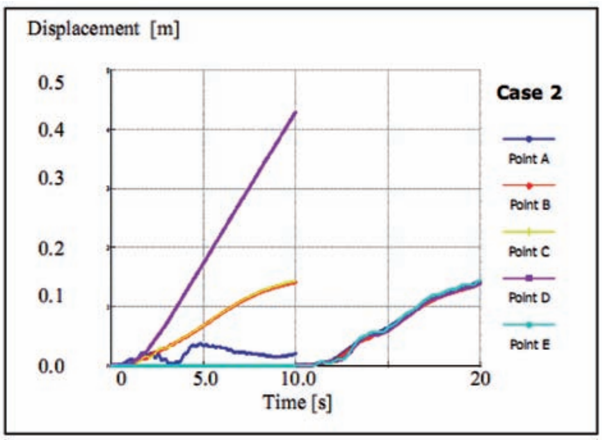

Fig. 9. Case 2, displacement time history in different points of the SSI system

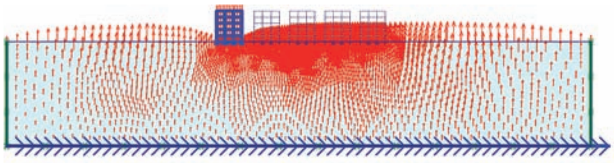

Fig. 10. Vertical displacements (Uy) – Extreme Uy=502.43 mm

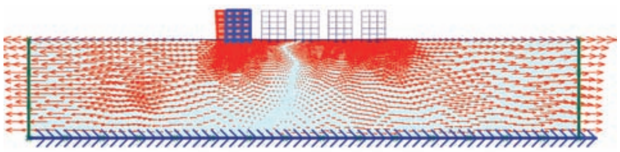

Fig. 11. Horizontal displacements (Ux) – Extreme Ux=95.63mm

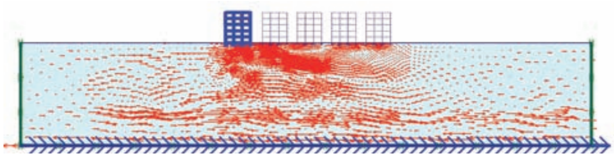

Fig. 12. Horizontal accelerations: Extreme horizontal acceleration 34 cm/s^2

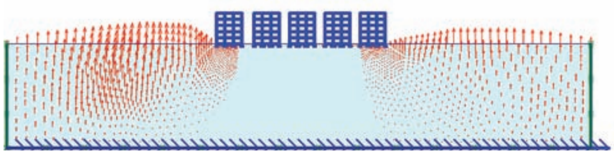

Fig. 13. Vertical displacements (Uy) – Extreme Uy=218.82 mm

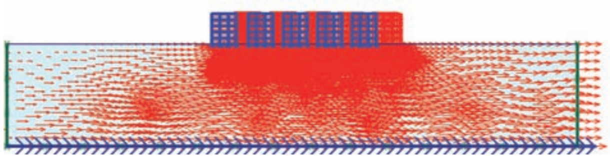

Fig. 14. Horizontal displacements (Ux): Extreme Ux = 147.85 mm

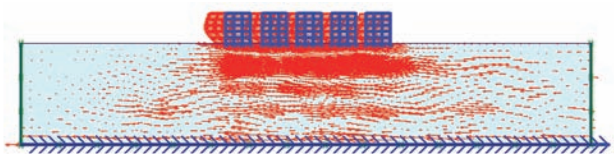

Fig. 15. Horizontal accelerations: Extreme horizontal acceleration 34 cm/s^2

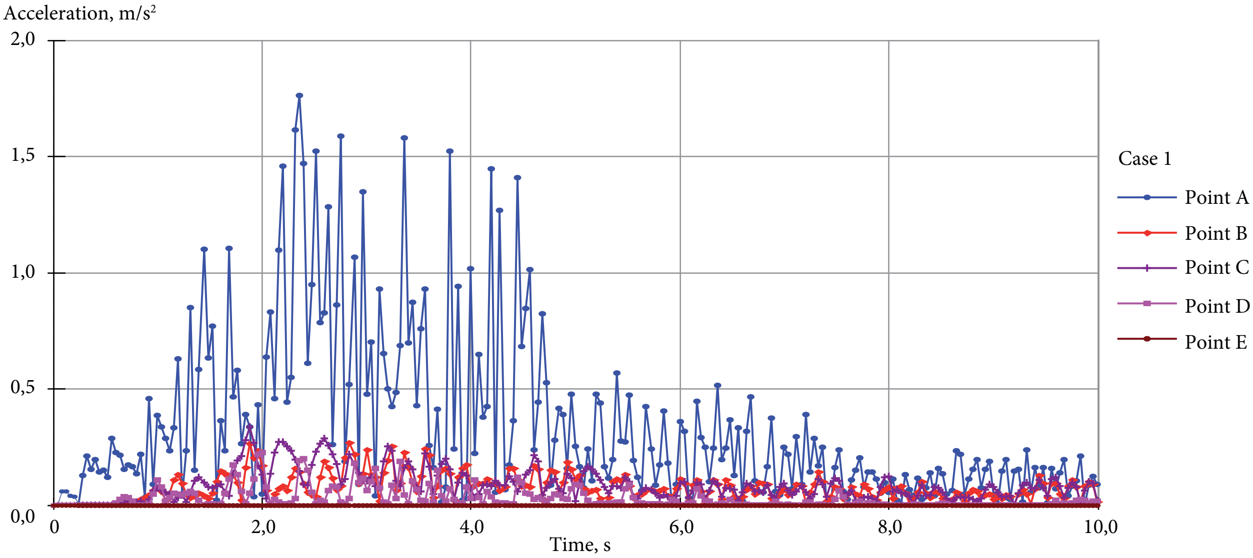

Fig. 16. Case 1, acceleration time history in different points of the SSI system

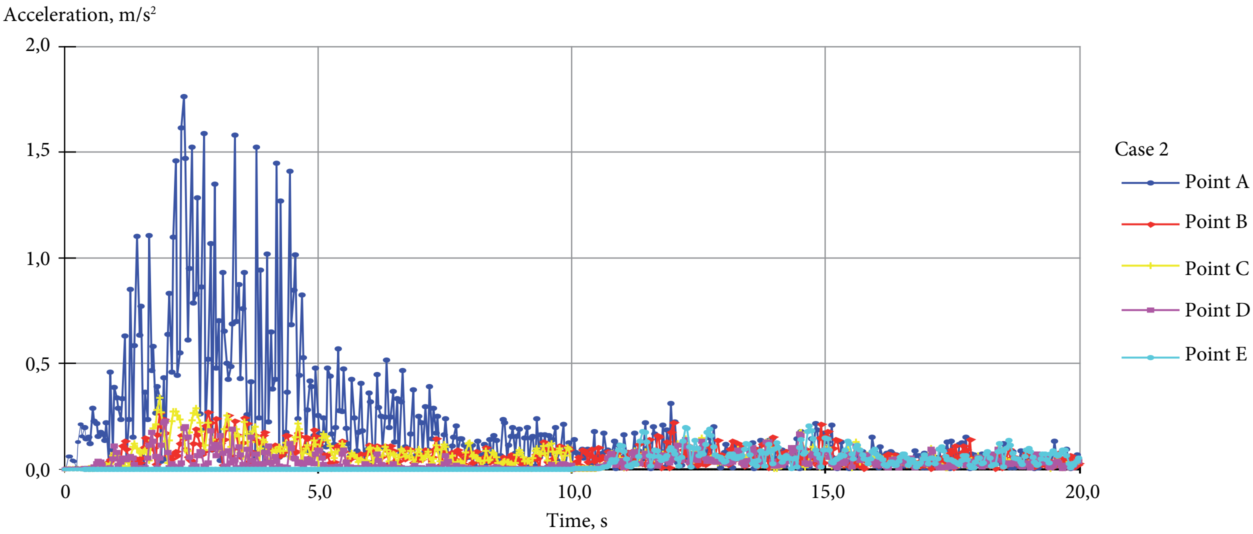

Fig. 17. Case 2, acceleration time history in different points of the SSI system

8. These results will have to be substantiated by more computational results with continuum or other exact methods to account for the damping effects, in-plane motion, and 3D models. In addition, it will be necessary to examine to what extent anomalous structural responses are affected by the type and location of the seismic source as well as by the pulse duration. Research work is in progress in our laboratory and will be published in the future.

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PASTATŲ GRUPĖS GRUNTO IR KONSTRUKCIJIŲ SĄVEIKOS ESANT SEISMINIAM POVEIKIUI ANALIZĖ

A. Gouasmia, K. Djeghaba

Santrauka. Pagal paskutiniu metu taikomą naują konstrukcijų, veikiamų seisminės apkrovos, skaičiavimo koncepciją, vadinamąją projektavimu, grįstu eksploatacinėmis savybėmis, būtina ypač atidžiai apsvarstyti atskirus aspektus analizuojant konstrukcijas. Vienas svarbiausių aspektų yra grunto ir konstrukcijos sąveika (SSI). Ji gali keisti dinamines konstrukcijos savybes, kurios gali būti palankios arba žalingos. Tiriant šį efektą, analizuota idealizuoto mažo miestelio, sudaryto iš penkių vienodai išdėstytų penkių aukštų monolitinių gelžbetoninių pastatų, seisminė priklausomybė. Pastatai užinkaruoti silpname grunte, kurio dalį sudaro ir tvirtos uolienos. Gauti rezultatai rodo gerą sutapimą bei papildo kitus gautus rezultatus šioje tyrimų srityje.

Reikšminiai žodžiai: grunto ir konstrukcijos sąveika, metodas, grįstas bangomis, statybos aikštelės efektas, seisminė priklausomybė, priklausomybės plėtotė.

Abdelhacine GOUASMIA. Doctor, Associate Professor. Dept. of Civil Engineering, University of Badji Mokhtar, Annaba Algeria. Engineer of civil engineering (1984). Master of Applied Science (1987). Doctor (2008). Research interests: soil-structure interaction, earthquake resistant design, seismic isolation, numerical analysis, soft computing.

Kamel DJEGHABA. Doctor, Associate Professor. Dept. of Civil Engineering, University of Badji Mokhtar, Annaba Algeria. Engineer of civil engineering (1984). Doctor (1990). Research interests: structural analysis of structures, earthquake resistant design, seismic isolation, numerical analysis, finite element method.