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INFLUENCE OF SOIL-STRUCTURE INTERACTION ON SEISMIC RESPONSE OF SHEAR BUILDINGS

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ABSTRACT

The beneficial or detrimental effect of dynamic Soil-Structure Interaction (SSI) is a controversial issue. With reference to an idealized R/C shear building, a comprehensive comparison between the results obtained with a classical fixed-base analysis and a complete SSI analysis is provided in the present paper.

The shear-type structure is modeled as generalized Single Degree Of Freedom (SDOF) system using the principle of virtual displacements. The foundation consists of surface square foundations resting on different soil conditions, consistent with the provisions of EC8-part I.

SSI effects in the far-field of earthquakes was evaluated by direct application of the elastic pseudo-acceleration/displacement design spectra proposed in EC8, taking into account the change in natural period and damping of the soil-structure system.

SSI effect in near-field area of earthquakes was analyzed using the computer program SASSI2000 (Lysmer et *al*., 1999), by means of a set of ten actual earthquakes recorded within a distance of 20 km from fault

The proposed analyses can be easily used by consultants who want to face the task of SSI in an immediate and simplified manner, without devoting resources into complex analysis. The charts proposed in this paper could be incorporated in seismic guidelines.

INTRODUCTION

Despite extensive research over the past 30 years, there is still controversy regarding the role of Soil-Structure Interaction (SSI) in the seismic performance of structures founded on soft soil. Neglecting SSI effects is currently being suggested in many seismic codes (ATC-3, NEHRP-97), as a conservative simplification that supposedly leads to improved safety margins (Mylonakis & Gazetas, 2000).

The interest in studying seismic SSI is motivated by the necessity of computing the modified seismic behavior of important structures, both in terms of stresses and strains. Such modifications could, in some cases, produce a detrimental effect on structures, as showed by many documented evidences around the world, e.g. during Mexico City (1985), Kobe (1995) and Bucharest (1977) earthquakes.

In general, a rigorous assessment of seismic SSI is not a simple task because of the difficulties associated with the evaluation of kinematic and inertial interaction effects in the structure and the soil. These analyses can be highly complex and require knowledge which is not offered in undergraduate civil engineering curricula. Therefore, solution to this problem that are capable of offering a satisfactory trade-off between rigor and simplicity would be desirable, especially in standard engineering practice.

To attain this aim, a systematic application of complete seismic SSI analyses to different types of buildings (up to twenty storeys), was performed; the compliance of the ground was evaluated by means of the computer program SASSI2000 (Lysmer et al., 1999). Concrete shear-type structures were modeled as generalized Single Degree Of Freedom (SDOF) systems using the principle of virtual displacements, while square surface footings resting on different soil conditions consistent with EC8-I are taken into account. The modified characteristics of the buildings, in terms of modified damping and period, were estimated using a recently published exact procedure (Maravas et al., 2007). Results are presented in form of ready-to-use non-dimensional charts.

The second objective of this work is the evaluation of SSI effects in terms of maximum displacements/accelerations at the top of buildings. A systematic comparison with the fixed-base solutions was performed. The goal is the set-up of simplified charts and tables that can be easily used by consultants who face the task of incorporating SSI in an immediate yet simplified manner, without performing expensive and time-consuming, analyses. Such tool could be useful for engineers, especially concerning the design of medium-rise reinforced-concrete buildings and/or for pre-design stages, where the SSI effect must be estimated and cannot be excluded a priori.

DYNAMIC SOIL-STRUCTURE INTERACTION

When subjected to dynamic loads, foundations oscillate in a way that depends on the nature and compliance of the supporting ground, the geometry and inertia of the foundation and superstructure, and the nature of dynamic excitation. Such an excitation may be in the form of a support motion due to waves arriving through the ground during an earthquake, an adjacent explosion, or the passage of a train; or it may result from the dynamic forces imposed directly or indirectly on the foundation from operating machines, ocean waves, and vehicles moving on the top of the structure (Gazetas, 1983).

In this paper the analyses will be focused on the behavior of different structures subjected to earthquake ground shaking.

It is widely recognized that the dynamic response of a structure supported on soft soil may differ substantially in amplitude and frequency content from the response of an identical structure supported on firm ground. Two main factors are responsible for this difference:

1. The flexibly-supported structure has more degrees of freedom and, consequently, different dynamic characteristics than the rigidly-supported (fixed base) structure;
2. A significant part of the vibrational energy of the flexibly-supported structure may be dissipated by radiation waves into the supporting medium or by damping in the foundation soil.

Note that there is no counterpart of the latter effect in a fixed base structure (Veletsos & Meek, 1974).

A key-step in carrying out such response analyses is to estimate the dynamic "spring" and "dashpot" coefficients of the flexibly-supported foundations.

The substructure approach is widely used in order to perform seismic SSI analyses. In such method the SSI problem is divided into two distinct parts which are combined to formulate the complete solution. The superposition inherent to this approach requires an assumption of linear soil and structural behavior.

A general procedure for the substructure approach can be developed in the realm of the following consecutive steps:

1. A Kinematic Interaction analysis, in which the foundation-structure system is assumed to have stiffness but no mass.
2. The foundation motion derived from the above analysis is used as input motion to the dynamic analysis of the superstructure modeled as a system on flexible base (Inertial Interaction Analysis). Foundation and structure are assumed to have stiffness and mass.

The most important geometric and material factors affecting the dynamic impedance of a foundation appear to be (Mylonakis et al., 2006):

1. Foundation shape (i.e., circular, strip, rectangular, arbitrary);
2. Type of soil profile (i.e., deep uniform or multi-layer deposit, shallow stratum on rock);
3. Foundation embedment (i.e., surface foundation, embedded foundation, pile foundation).

For a project of critical significance, a case-specific analysis must be performed using the most suitable numerical computer program. Natural soil deposits are frequently underlain by very stiff material or bedrock at shallow depth (H), rather than extending to a practically infinite depth as the homogeneous halfspace assumption implies. The proximity of such stiff formation to the oscillating surface modifies the static stiffness, K, and dashpot coefficients, $C(\omega)$. In particular, the static stiffnesses in all modes decrease with the relative depth to bedrock H/B (with B being the characteristic length of the foundation).

Effect of SSI

The classical approach for elasto-dynamic analysis of Soil-Structure Interaction aims at replacing the actual structure by an equivalent simple oscillator supported on a set of frequency-dependent springs and dashpots accounting for the stiffness and damping of the compliant soil-foundation system.

The system studied is shown in Fig. 1 and 2. It involves a simple oscillator on flexible base representing a single storey structure, or a multi storey structure after a pertinent reduction of its degrees-of-freedom (e.g., considering that the mass is concentrated at the point where the resultant inertial force acts).

The structure (Fig. 1) is described by its stiffness k, mass m, height h, and damping ratio ξ, which may be either viscous or linearly hysteretic. The foundation consists of a rigid surface squared footing of characteristic length B, resting on a homogeneous, linearly elastic, isotropic halfspace described by its shear modulus G_s, mass density ρ_s, Poisson's ratio ν_s, and hysteretic damping ratio ξ_s. The translational and rotational stiffness, K_x and K_r respectively, of the compliant soil-foundation system, is modeled by a pair of frequency-dependent springs. To ensure uniform units in all stiffness terms, K_r is represented by a translational vertical spring acting at distance r from the center of the footing.

The translational and rotational damping, C_x and C_r respectively, of the compliant soil-foundation system, is modeled by a pair of frequency-dependent dashpots, attached in parallel to the springs, representing energy loss due to hysteretic action and wave radiation in the soil medium. In this first step, the influence of foundation embedment and foundation mass is neglected.

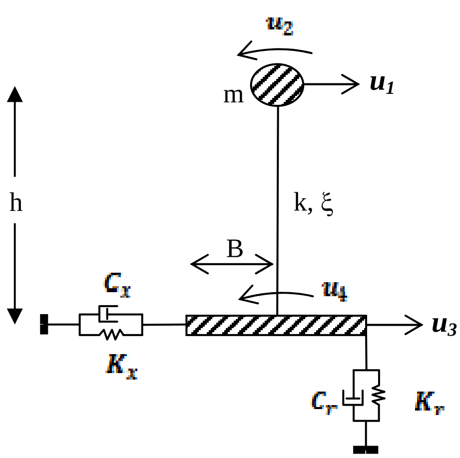

Fig. 1: Structure idealized by a stick model
(after Veletsos, 1977 and Maravas et al., 2007)

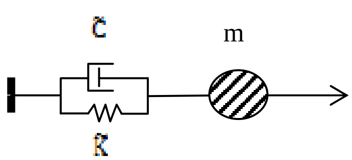

Fig. 2: Reduced single degree-of-freedom model
(after Veletsos, 1977 and Maravas et al., 2007)

In the present paper a systematic application of the exact solution recently proposed by Maravas et al. (2007) is presented (Equation 1 and 2).

The method contains no approximations in the derivation of the fundamental natural period, $\tilde{\omega}$, and effective damping, $\tilde{\xi}$, of the system. Furthermore, the exact frequency-varying foundation impedances may be employed.

The properties of the replacing oscillator (Fig. 2) are given by:

$$\tilde{\xi} = \frac{\dfrac{\xi_x}{\omega_x^2\left(1+4\xi_x^2\right)} + \dfrac{\xi_r}{\omega_r^2\left(1+4\xi_r^2\right)} + \dfrac{\xi}{\omega_c^2\left(1+4\xi^2\right)}}{\dfrac{1}{\omega_x^2\left(1+4\xi_x^2\right)} + \dfrac{1}{\omega_r^2\left(1+4\xi_r^2\right)} + \dfrac{1}{\omega_c^2\left(1+4\xi^2\right)}} \tag{1}$$

$$\tilde{\omega}^2 = \left[\frac{1+4\tilde{\xi}^2}{\omega_x^2\left(1+4\xi_x^2\right)} + \frac{1+4\tilde{\xi}^2}{\omega_r^2\left(1+4\xi_r^2\right)} + \frac{1+4\tilde{\xi}^2}{\omega_c^2\left(1+4\xi^2\right)}\right]^{-1} \tag{2}$$

where $\xi_i = \dfrac{\overline{\omega} C_i}{2K_i}$, ($i$ = x, r) with $\overline{\omega}$ being the circular excitation frequency.

<u>Dimensionless parameters</u>

As mentioned before, the response of the foundation-structure system depends on the properties of the foundation and the supporting medium, the properties of the superstructure and the characteristics of the excitation. The effects of these factors on SSI can best be expressed in terms of dimensionless parameters, defined by the following equations:

1. wave parameter, $1/\sigma$

$$\frac{1}{\sigma} = \frac{h}{T\, V_S} \tag{3}$$

where T denotes the natural period of the fixed-base structure and V_S is the shear-wave velocity of the soil.

The wave parameter $1/\sigma$ may be looked upon as a measure of the relative stiffness of the soil foundation and the superstructure.

2. slenderness ratio, h/B, where B is the characteristic length of the foundation base.

3. frequency parameter, a_0, given by :

$$a_0 = \frac{\overline{\omega} B}{V_S} \tag{4}$$

4. relative mass density for the structure and the supporting soil, γ

$$\gamma = \frac{m}{\pi \rho_s h B^2} \tag{5}$$

5. Ratio of the foundation mass to the mass of the superstructure, μ

$$\mu = \frac{m_b}{m}$$

6. Damping ratio of the structure for fixed base conditions, ξ.

7. Poisson's ratio of the soil, ν_s.

8. Hysteretic damping ratio of the soil, ξ_s.

For typical frequencies generated during earthquake shakings, dimensionless frequency parameter, a_0, ranges between 0 and 2.

GENERALIZED SDOF SYSTEMS

The analysis of a complex system can be approximated using an equivalent Single Degree of Freedom (SDOF) system; such system is called a generalized SDOF system.

The analysis provides exact results for an assemblage of rigid bodies supported such that it can deflect in only one shape, but only approximate results can be obtained for systems with distributed mass and flexibility. In the latter case, the approximate natural frequency is shown to depend on the assumed deflected shape (Chopra, 1995).

In both categories, the structure is forced to behave like a SDOF system by the fact that displacements of only a single form or shape are permitted; the assumed deflection can be related to a single generalized displacement z(t) through a shape function $\psi(x)$ that approximates the fundamental vibration mode and can be expressed as:

$$u(x,t) = \psi(x) \cdot z(t) \tag{6}$$

The equation of motion for a generalized SDOF system can be written as:

$$\tilde{m}\ddot{z} + \tilde{c}\dot{z} + \tilde{k}z = \tilde{p}(t) \tag{7}$$

where $\tilde{m}, \tilde{c}, \tilde{k}$ and $\tilde{p}(t)$ are the generalized mass, generalized damping, generalized stiffness and generalized excitation, respectively. These generalized properties are associated with the selected generalized displacement z(t) (Chopra, 1995).

The aim of the present paper is the evaluation of the response of generalized SDOF systems representing different configurations of shear buildings. An estimation of the generalized properties of lumped-mass systems (Fig. 3) is performed in the ensuing.

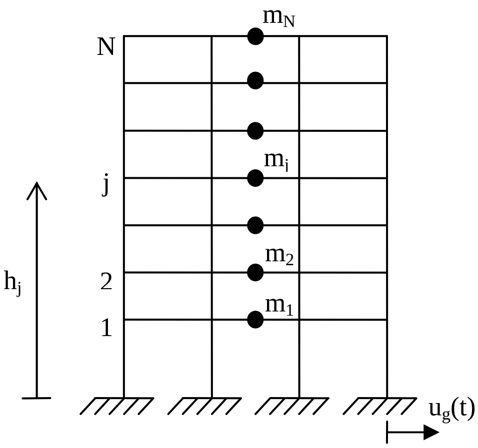

Fig. 3: Lumped-mass system - Shear building

We assume that the floor displacements relative to ground can be expressed as:

$$u_j(t) = \psi_j \cdot z(t) \qquad j = 1,2,\ldots,N \tag{8}$$

where ψ_j is an assumed shape vector that defines the deflected shape of the system. The total displacement of the j^{th} floor is

$$u_j^t(t) = u_j(t) + u_g(t) \tag{9}$$

The equation of motion of the generalized SDOF system can then be expressed as:

$$\tilde{m}\ddot{z} + \tilde{k}z = -\tilde{L}\ddot{u}_g(t) \tag{10}$$

where

$$\tilde{m} = \sum_{j=1}^{N} m_j \cdot \psi_j^2 \qquad \tilde{k} = \sum_{j=1}^{N} k_j \cdot \left(\psi_j - \psi_{j-1}\right)^2$$

$$\tilde{L} = \sum_{j=1}^{N} m_j \cdot \psi_j$$

An approximate shape function $\psi(x)$ may be determined as the deflected shape due to static forces $p(x) = m(x) \cdot \tilde{\psi}(x)$, where $\tilde{\psi}(x)$ is any reasonable approximation of the exact deformation shape.

One common selection for these forces is the weight, w_j, of each storey applied in an appropriate direction. The displacement u_j and force boundary conditions are satisfied automatically if the shape function is determined from the static deflection due to a selected set of forces. This concept is very useful for lumped-mass systems (Chopra, 1995).

According to this criterion, the set of forces shown in Fig. 4 was used to determine the equivalent fundamental frequency:

$$\omega_n = \sqrt{\frac{g\sum_{j=1}^{N} m_j \cdot u_j}{\sum_{j=1}^{N} m_j \cdot u_j^2}} \qquad (11)$$

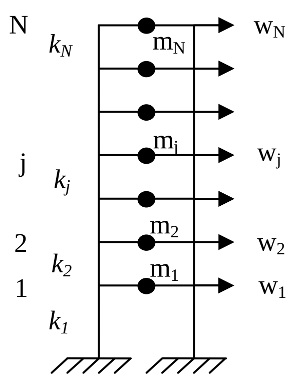

Fig. 4: Shape function from deflection due to static forces

PERFORMED ANALYSES AND RESULTS

The analyses were performed with reference to a set of simple shear-building configurations resting on different soil deposits and subjected to different seismic input motions.

A single value of peak ground acceleration, a_g, of 0.35g was selected for all the analyses and different configurations were assumed for:

1. soil class
2. building type
3. seismic input (far- and near-field motion)

Soil class

For the foregoing analyses, three different homogeneous soil deposits characterized by the following shear-wave velocities, V_S, was selected:

- V_S = 80 m/s
- V_S = 200 m/s
- V_S = 320 m/s

The homogeneous halfspace configuration was considered, in addition to 4 more different cases, based on the bedrock depth, i.e.:

- Bedrock at 5m depth
- Bedrock at 10m depth
- Bedrock at 20m depth
- Bedrock at 50m depth

giving a total number of 15 cases analyzed.
Soil damping, ξ_s, was assumed equal to 5%.

The considered subsoil conditions correspond to three different soil classes, according with EC8 (Table 1).

Table 1: Site characterization

a_g [g]	Subsoil conditions	Soil Class (EC8-I)
0.35	V_S = 80 m/s (Halfspace and Bedrock 50m depth)	D
	V_S = 200 m/s (Halfspace and Bedrock 50m depth)	C
	V_S = 320 m/s (Halfspace and Bedrock 50m depth)	
	V_S = 80 m/s (Bedrock from 5 to 20m depth)	E
	V_S = 200 m/s (Bedrock from 5 to 20m depth)	
	V_S = 320 m/s (Bedrock from 5 to 20m depth)	

Building types

Different ordinary concrete shear-building configurations were selected in order to perform the analyses; a general 3D model of the buildings under investigation is showed in Fig. 5.

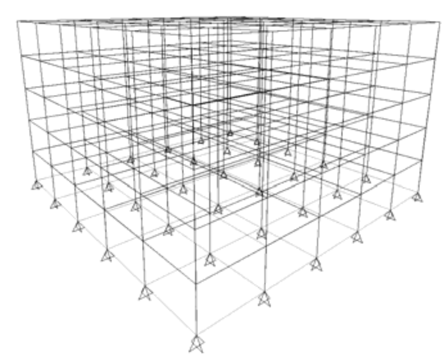

Fig. 5: General 3D model

The analyses were restricted to twelve surface squared founded building configurations, such as:

- number of storeys: 2, 5, 10, 20
- number of bays: 2x2, 5x5, 10x10

A storey height H = 3m and a bay length B = 5m were assumed. The 2D model is represented in Fig. 6

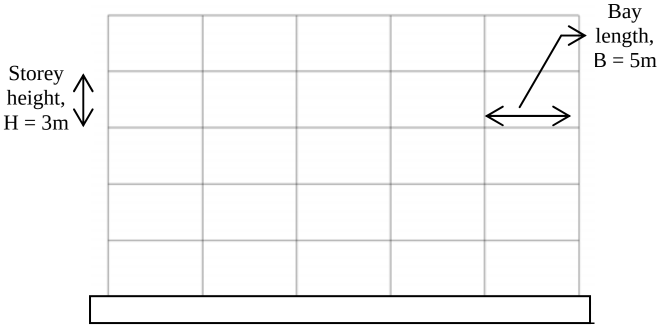

Fig. 6: General 2D model

<u>Seismic input</u>

The herein reported study focuses on the comparison of the results in term of pseudo-spectral accelerations, S_a, and displacements, S_d, obtained from the fixed-base and SSI configurations of the concrete shear-buildings under investigations.

As specified, the presence of deformable soil supporting a structure affects its seismic response in many different ways. Firstly, a flexibly-supported structure has different vibrational characteristics, most notably a longer fundamental period, $\tilde{T}$, than the period T of the corresponding rigidly-supported (fixed-base) structure. Secondly, part of the energy of the vibrating flexibly-supported structure is dissipated into the soil through wave radiation and hysteretic action, leading to an effective damping ratio, $\tilde{\xi}$, which is usually larger than the damping ξ of the corresponding fixed-base structure (Mylonakis & Gazetas, 2000).

Consequently, the seismic design of structures supported on deformable ground must properly account for such an increase in fundamental period and damping.

With little exception (e.g. NZS4203), seismic codes today use idealized smooth design spectra which attain constant acceleration up to a certain period and thereafter decrease monotonically with period. As a consequence, consideration of SSI leads invariably to smaller accelerations and stresses in the structure and its foundation.

On the other hand, the increase in period due to SSI leads to higher relative displacements which, in turn, may cause an increase in seismic demand.

SSI effects in far-field were estimated with reference to the elastic pseudo-acceleration/displacement design spectra proposed in EC8-I, after having evaluated the change in natural period and damping of soil- structure system. Values of h/r ratio ranging from 0.2 to 8.2 and wave parameter ($1/\sigma$) ranging from 0.05 to 0.69 were considered in the analyses.

SSI effects in near-fault were analyzed with reference to a set of different actual near-fault earthquakes, recorded within 20 km distance. Since only low shear-wave velocities (V_S = 80 m/s and 200 m/s) were considered, wave parameter ranges between 0.12 and 0.69. As for the analyses in far-field, values of h/r ratio ranging from 0.2 to 8.2 were considered.

<u>Results in terms of modified natural period</u>

In figures 7 to 10, the dimensionless parameter $\tilde{T}/T$ is plotted as a function of $1/\sigma$ for all bedrock configurations considered.

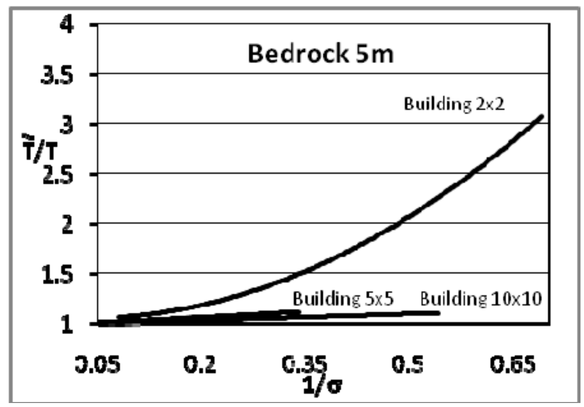

Fig. 7: Modified damping for 5m-deep bedrock

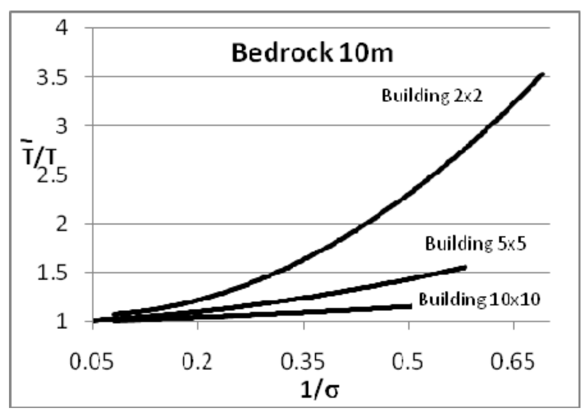

Fig. 8: Modified damping for 10m-deep bedrock

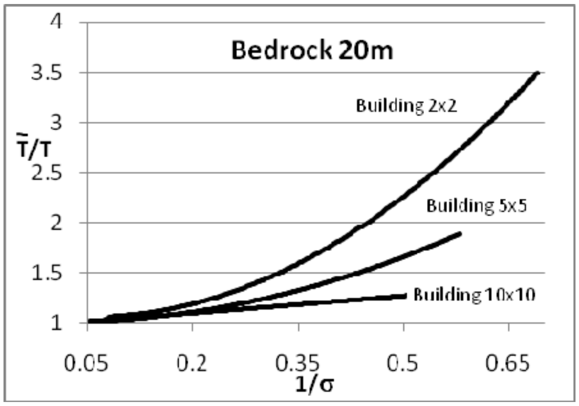

Fig. 9: Modified damping for 20m-deep bedrock

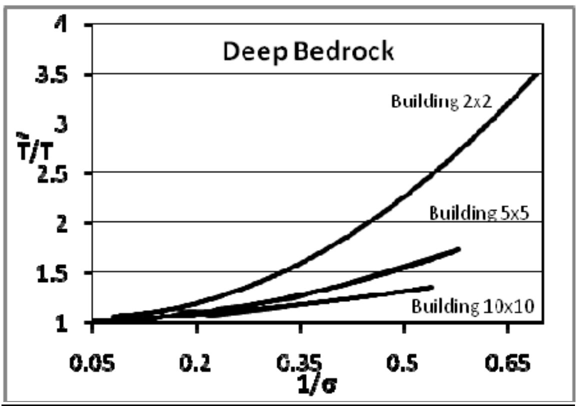

Fig. 10: Modified damping for deep bedrock

As figures 7 to 10 evidence, $\tilde{T}/T$ ratio increases considerably with 1/σ for building 2x2 bays, whereas it becomes less significant for buildings of major dimensions (5x5 and 10x10 bays). Moreover, increasing is smaller for shallow bedrock (5 m) than for deep bedrock and the difference between 10, 20, 50 m depth are not relevant.

Note that solution for 50m bedrock depth and halfspace showed no practical differences and they are reported in the same chart (Fig. 10).

<u>Results in term of Pseudo-Spectral acceleration/displacement (far-field)</u>

The outcomes of the analyses show some expected evidences:

1. A systematic reduction in the seismic demand in term of pseudo spectral accelerations, S_a; such effect appear more evident for deep bedrock (soil class C and D) configurations.
   Such changing are more pronounced for
   - softer soils
   - stiffer structures
   - taller structures

Average results are resumed in tables 2, 3 and 4:

Table 2: Reduction of S_a as function of soil stiffness

V_S [m/s]	S_a average reduction [%] (deep bedrock Soil class C and D)	S_a average reduction [%] (shallow bedrock Soil class E)
80	39.6	30.4
200	16.9	12.6
320	9.1	5.3

Table 3: Reduction of S_a as function of structure stiffness

Building type	S_a average reduction [%] (deep bedrock Soil class C and D)	S_a average reduction [%] (shallow bedrock Soil class E)
2x2	28.7	26.2
5x5	15.3	10.3
10x10	12.5	4.9

Table 4: Reduction of S_a as function of structure tallness

Storeys #	S_a average reduction [%] (deep bedrock Soil class C and D)	S_a average reduction [%] (shallow bedrock Soil class E)
2	6.6	4.6
5	7.1	5.6
10	14.7	11.9
20	28.2	20.4

2. A general increase in the seismic demand in terms of pseudo spectral displacements, S_d; such effect appear to be independent on the bedrock configurations.
   In this case, such changes are more pronounced for
   - softer soils
   - stiffer structures
   - taller structures

Average results are resumed in tables 5, 6 and7:

Table 5: Increasing of S_d, as function of soil stiffness

V_S [m/s]	S_d average increase [%]
80	23.9
200	3.0
320	0.9

Table 6: Increasing of S_d, as function of structure stiffness

Building type	S_d average increase [%]
2x2	24.1
5x5	3.2
10x10	0.7

Table 7: Increasing of S_d, as function of structural height

Storeys #	S_d average increase [%]
2	3.4
5	5.0
10	6.0
20	13.3

Results obtained from the parametric analysis can be generalized to be useful for design purposes; in particular the acceleration, $\tilde{S}_a/S_a$, and displacement, $\tilde{S}_d/S_d$, ratios, are plotted as function of $1/\sigma$, μ and h/r.
Whenever necessary, results were sub-divided for the different soil classes.

Regarding to the obtained results, some general remarks on the findings can be done:

1. A systematic reduction of the ratio $\tilde{S}_a/S_a$ with $1/\sigma$ can be noted; such reduction is more evident for deep bedrock configurations, i.e. soil class C and D, for tall structures, i.e. high ratio h/r, and for massive superstructure, i.e. low ratio μ..
Differences in the results of no practical interest are revealed for h/r less than 2.

For soft soils and tall structures, the ratio $\tilde{S}_a/S_a$ shows a substantial reduction, of the order of 80% or so.

Good agreement is shown in the experimental results (see Fig. 11 and 12).

2. A systematic increase of the ratio $\tilde{S}_d/S_d$ with $1/\sigma$ is observed; relevant rising is revealed especially for $1/\sigma$ greater than 0.25 and shows high value for h/r greater than 4 and for μ less than 0.5 (see Fig. 13).

Not relevant differences are evidenced from deep bedrock and shallow bedrock configurations.

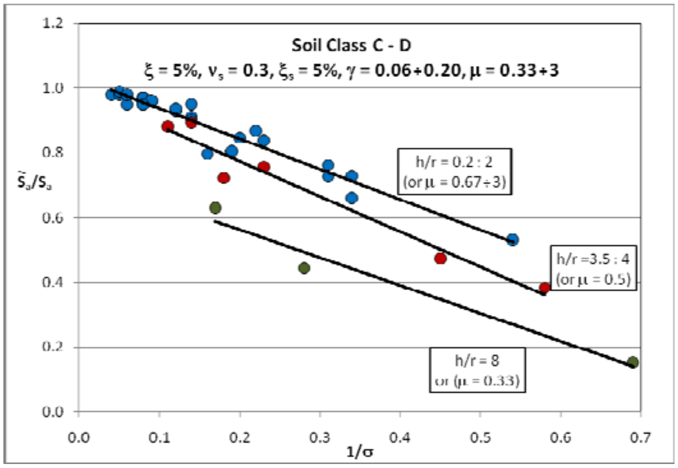

Fig. 11: $\tilde{S}_a/S_a$ *as function of 1/σ and h/r (soil class C - D)*

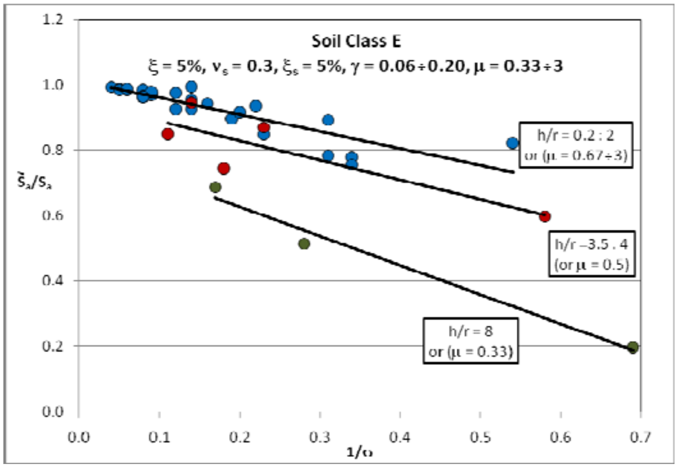

Fig. 12: $\tilde{S}_a/S_a$ *as function of 1/σ and h/r (soil class E)*

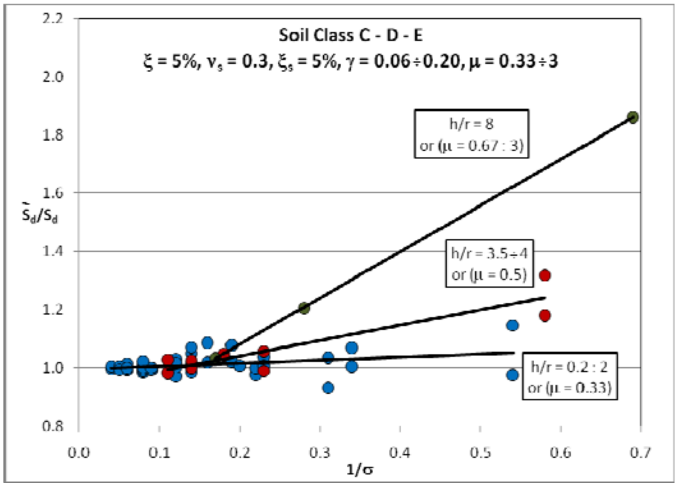

Fig. 13: $\tilde{S}_d / S_d$ as function of 1/σ and *h/r* (soil class *C-D-E*)

<u>Results in term of Pseudo-Spectral acceleration/displacement (near-field)</u>

A site located close to the source of a seismic event may be in a geometrical configuration, in respect to the propagating rupture, which may favor the constructive interference of waves (synchronism of phases causing building up of energy) traveling to it, which may result in a large velocity pulse, that clearly distinguish them from typical far-field ground motions. This situation, for dip-slip faults, requires the rupture going toward the site and the alignment of the latter with the dip of the fault, whereas for strike-slip faults the site must be aligned with the strike; if these conditions are met the ground-motion at the site may show forward directivity effects (Somerville, 1997); such effect, may be an important contributing factor in the large spectral values at T > 0.50s in near-fault seismic motions. The propagation of fault rupture toward a site at very high velocity causes most of the seismic energy from the rupture to arrive in a single long-period pulse of motion, at the beginning of the recording. The effect of forward rupture directivity on the response spectrum is to increase the spectral values of the horizontal component normal to the fault strike at periods longer than about 0.5s.

Common record selection practice does not apply in the near-source.

The near-fault strong ground motion database we have used for the analysis consists of 10 processed near-field strong ground motion records from a variety of tectonic environments.

Evidently, records with enhanced spectral ordinates at large periods are not rare in nature, whether due to soil or seismological factors.

It is therefore apparent that as a result of soil or seismological factors, an increase in the fundamental period due to SSI may lead to increased response (despite a possible increase in damping), which contradicts the expectation incited by the conventional design spectrum (Mylonakis & Gazetas, 2000).

The database of actual recorded ground-motion time histories, from different fault types (i.e., strike-slip, reverse, oblique) and earthquake magnitudes (i.e., M_w 5.6–6.7), was compiled from well known and extensively studied seismic events:

- Parkfield, CA, USA (Station CO2)
- San Fernando, CA, USA (Station PCD)
- Coyote Lake, CA, USA (Station GA6)
- Imperial Valley, CA, USA (Station E07)
- Morgan Hill, CA, USA (Station CLD)
- Nahanni, Canada (Station SITE1)
- Palm Spring, CA, USA (Station NPS)
- Whittier Narrows, CA, USA (Station DOW)
- Superstition Hills, CA, USA (Station, ELC)
- Erzincan, Turkey (Station ERZ)

All the motions were recorded at stations located within 20 km from the causative fault and distinct strong velocity pulses are recognized, with the only exception of Nahanni earthquake.

The analyses performed in the present paper consist in the application of all the earthquakes listed above to the Soil-Structure configurations previously defined; the analyses were restricted to 2x2 buildings and to soil deposits with low shear-wave velocity, i.e. V_S = 80-200 m/s (Soil Class C – D). Such Soil-Structure configurations were selected because of their high susceptibility to the effects imposed by Soil-Structure Interaction analysis.

Results presented in tables hereinafter have not to be considered as general outcomes of near-fault earthquakes application. A generalization is well above the scope of the present paper and, for the knowledge of the authors, such an attempt of generalization is inappropriate in the field of near-fault effects. Many parameters have to be carefully analyzed by seismologists, geologists and engineers, e.g. characteristics of the causative fault, geology of the deposit, path of the travelling waves, building typology.

Nevertheless, the analyses performed have showed some interesting results that, if carefully interpreted, could lead some general understandings of the phenomenon.

In the following tables results obtained from different near-fault earthquake for the fixed-base solution, in term of Spectral Accelerations, S_a, and Displacements, S_d, at the top of the SDOF systems and the SSI solutions in term of $\tilde{S}_a$ and $\tilde{S}_d$ are compared.

In order to assess whether taking into account SSI effects might lead to a detrimental effect in the seismic demand of structures, results are presented in the form of $\tilde{S}_a / S_a$ and

$\tilde{S}_d / S_d$ is sub-divided for deep and shallow bedrock presence.

The analyses were performed using the computer program SASSI2000 (Lysmer at al., 1999)

From the results of the 400 analyzed configurations, it is evident that the general trend suggested by EC8-I, where the reduction of the Spectral Acceleration is achieved if SSI analyses are performed, is not always confirmed. In fact, in some cases, S_a increases significantly, as shown in Table 8.

Table 8: Increase of S_a

Increase [%]	h/r	Bedrock	Soil class	Earthquake ID
16	2.1	Deep	D	Morgan Hill
13	2.1	Shallow	D	Morgan Hill
12	4.1	Deep	D	Imperial V.
18	8.2	Deep/Shallow	D	San Fernando
16	0.9	Shallow	C	San Fernando

Generally we observed that the reduction of S_a is more often achieved for Soil Class D (68% of the cases) than for Soil Class C (39%) and is more pronounced for shallow bedrock configurations; the latter observation is due to the complex resonance phenomena that could occur if the frequency of the excitation is close to the frequencies of the deposit and of the structure.

Concerning the Spectral Displacement, S_d, huge increases are showed, especially for Soil Class D and squat structures, as Table 9 evidences.

Table 9: Increase of S_d

Increase [%]	h/r	Bedrock	Soil class	Earthquake ID
79	0.9	Shallow	D	Parkfield
61	0.9	Shallow	D	San Fernando
40	0.9	Shallow	D	Coyote Lake
69	0.9	Shallow	D	Imperial V.
70	0.9	Shallow	D	Morgan Hills
70	0.9	Shallow	D	Erzincan

CONCLUSIONS

Analyses were performed to evaluate beneficial or detrimental effects of dynamic Soil-Structure Interaction (SSI) with reference to a set of reinforced concrete shear-type structures, taking into account different soil and seismic conditions.

At first, the modified characteristics of the buildings, in terms of modified damping and period were estimated. The dimensionless parameter $\tilde{T}/T$ was plotted in non-dimensional charts as a function of $1/\sigma$ for the different bedrock configurations considered. The above mentioned charts evidence that:

1. $\tilde{T}/T$ ratio increases significantly with $1/\sigma$ for building 2x2 bays and reaches values of 3-3.5; for buildings 5x5 and 10x10 bays we note a minor increase with maximum values of about 1-1.5;
2. $\tilde{T}/T$ ratio increase is smaller for shallow depth of the bedrock (5 m) than for deep bedrock and the difference between 10, 20, 50 m depth are not relevant.

In the second part of the study, SSI effects in far-field and near- field were evaluated in terms of maximum displacements/accelerations at the top of the buildings and a systematic comparison with the fixed-base solutions was performed. Results of far-field analyses were synthesized in non-dimensional charts, in order to be useful for design purposes. Regarding the obtained results, the following conclusions can be draw:

1. systematic reduction in terms of pseudo-spectral acceleration, S_a, and increase in terms of pseudo-spectral displacement, S_d, was evidenced;

2. the changes in seismic demand are more evident for softer soils, stiffer and taller structures;
3. spectral acceleration reduction appears more evident for deep bedrock configurations, while no relevant differences are evidenced from deep and shallow bedrock in spectral displacement increasing.

The results of the near-fault analyses show that Soil-Structure Interaction effect during near-fault ground motions is a complex phenomenon that require deep local seismological, geological and engineering investigations.

Therefore it is evident that SSI effect during near-fault events may lead to an increase of the seismic demand, both in terms of spectral acceleration and spectral displacement.

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