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A novel simple procedure to consider seismic soil structure interaction effects in 2D models

Juan Diego Jaramillo[1†], Juan David Gómez[1†], Doriam Restrepo[1†] and Santiago Rivera[2‡]

1. *Universidad EAFIT, Medellín, Colombia*

2. *INTEGRAL S.A, Medellín, Colombia*

Abstract: A method is proposed to estimate the seismic soil-structure-interaction (SSI) effects for use in engineering practice. It is applicable to 2D structures subjected to vertically incident shear waves supported by homogenous half-spaces. The method is attractive since it keeps the simplicity of the spectral approach, overcomes some of the difficulties and inaccuracies of existing classical techniques and yet it considers a physically consistent excitation. This level of simplicity is achieved through a response spectra modification factor that can be applied to the free-field 5%-damped response spectra to yield design spectral ordinates that take into account the scattered motions introduced by the interaction effects. The modification factor is representative of the Transfer Function (TF) between the structural relative displacements and the free-field motion, which is described in terms of its maximum amplitude and associated frequency. Expressions to compute the modification factor by practicing engineers are proposed based upon a parametric study using 576 cases representative of actual structures. The method is tested in 10 cases spanning a wide range of common fundamental vibration periods.

Keywords: seismic soil structure interaction; simplified procedures; modified response spectra

1 Introduction

Seismic soil-structure-interaction (SSI) has been intensively studied over the last four decades. A recent, detailed account of the historical development of the subject, starting from the early treatment of fundamental solutions under static and dynamic conditions for full and half-spaces and concluding with the current research approaches in terms of numerical techniques, is described in Kausel (2010).

Even after strong efforts to understand the physics of the problem, full SSI studies still remain restricted to sensitive engineering projects like dams, power plants and bridges. In contrast, the effects of SSI are seldom considered in the evaluation of the earthquake response of moderate projects like residential buildings. At the same time, there is the widespread belief among the engineering community that neglecting SSI effects generally results in conservative designs. This scarce consideration of SSI effects on standard projects may be linked to methodologies limited in scope and/or too complex to be used on a routine basis. In this work, the problem of SSI effects in terms of an engineering method that overcomes some of the inaccuracies and existing complexities of current techniques is addressed, while keeping the physical basis of the problem.

As can be inferred from the contribution by Kausel (2010), the simplified models leading to the current seismic code procedures are based on idealizations originally proposed by Sezawa and Kanai (1935), Parmlee (1967), Sarrazin (1970), Sarrazin *et al*. (1972) and concluding with the consolidation of the equivalent oscillator method proposed by Jennings and Bielak (1973), Veletsos and Meek (1974), Veletsos and Nair (1975) and Veletsos (1977). Since then, most of the available simplified techniques for SSI effects considered in seismic provisions have remained unmodified, leading to a lack of consideration or even understanding of the problem itself.

One of the available strategies, described in Clough and Penzien (1993), and based on contributions by Veletsos and Wei (1971), Luco and Westman (1971), Veletsos and Meek (1974) and Gazetas (1983) among others, uses substructuring ideas in the form of a two-step procedure. That approach requires, in a first analysis step, the computation of the so-called impedance or frequency dependent dynamic stiffness functions $K(i\omega)$ of the soil-foundation system (Substructure 1). The impedance functions, together with the dynamic properties of the superstructure (Substructure 2), are then used in a second step to establish a system of frequency domain equations, where the displacements from

Correspondence to: Juan Diego Jaramillo, Universidad EAFIT, Cra. 49 #7sur-50, Medellín, Colombia
Tel: +57 (574) 2619353
E-mail: jjarami@eafit.edu.co
† Professor; ‡ Consulting Engineer

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both foundation and structure form a set of unknown variables. In this technique, the foundation system is implicitly assumed to consist of a rigid massless plate resting on top of a homogeneous half-space, while the excitation corresponds to equivalent forces, computed from a prescribed free-field motion. Although this procedure is rigorous for the case of surface foundations, it is also far from being usable during daily practice due to its frequency domain basis, which is resisted by most engineers at the practical level.

A second approach, which forms the basis of most code regulated techniques, is the one originally proposed by Jennings and Bielak (1973), Veletsos and Meek (1974), Veletsos and Nair (1975), Veletsos (1977). The method takes advantage of the fact that the transfer function (TF) of the interacting system, or ratio between the structural displacements relative to the base $V(\mathrm{i}\omega)$, and the free-field motion at the soil surface $V^0(\mathrm{i}\omega)$, is similar in shape to the one of a fixed-base single degree of freedom (SDOF) system. Accordingly, the analyst determines the seismic demands from a conventional design spectra, but accounting for the SSI effects in terms of a modified damping ratio and vibration period, thus treating the problem as in the conventional case of a fixed-base oscillator. The equivalent oscillator method is very appealing in engineering applications, since it keeps the procedure within the context of the classical fixed-base system, although it is out of the actual physical context of the SSI problem. The main complexity of the method lies in the procedure to obtain the modified damping and vibration period of the equivalent SDOF-system, since both parameters must be obtained using the frequency dependent dynamic stiffness or impedance functions of the foundation. This fact creates the need to perform iterations, so the technique becomes cumbersome for use in practice.

An alternative but oversimplified way-around this problem, which is incorporated in most codes such as NEHRP (2009), eliminates the frequency dependence of the impedance functions by using an average value and introducing a simplified expression to obtain the modified damping ratio. Other seismic codes, like the NTC (2004), evaluate the impedance functions at the natural vibration period of the fixed-base structure. One drawback of the equivalent oscillator method is the fact that the actual TF in an interacting system can be shown to substantially differ from the usual shape for a fixed-base structure. This difference is clear in cases where the SSI effects are strong, e.g., when the damping and natural frequency of the system are far from those in a fixed-base structure.

In this work, simplified practice-oriented approach to consider SSI effects that overcomes some of the inherent complexities in the above two procedures is proposed. The two-fold technique aims at the direct computation of the modified SSI design response spectral ordinate based on a description of the TF between the relative structural displacements and the free-field motion. For this purpose, the TF is characterized in terms of its main controlling parameters, namely; its maximum amplitude and associated frequency, which are subsequently used to compute a response spectra modification factor to be imposed over the free-field 5%-damped response spectra. Two expressions are proposed to derive the TF parameters, which are strictly dependent on standard geometric and dynamic properties of the structure-foundation system. These expressions yield results closely related to the equivalent damping and vibration period of Jennings and Bielak's method. In the current work however, these parameters are called the TF characteristic parameters. This is in order to make clear that in contrast with the two classical approaches described above, this last function considers the fact that when the SSI effects are strong, the actual TF and that of an equivalent fixed-base SDOF-system exhibits substantial differences. In order to propose expressions for the determination of the TF characteristic parameters, a parametric study is performed using a boundary element method based numerical algorithm (BEM). To show the benefits of the technique, the simplified procedure is applied in the computation of a modified design spectra for a range of oscillators resting on a half-space-foundation system.

In the first part of the paper, the idea behind the proposed method is described and the range of variation of the involved parameters is defined. In the same section, a numerical validation of the in-house software used to conduct the parametric analysis is presented. Then, TFs corresponding to three extreme but relevant cases are discussed. Since the proposal is largely based on results obtained for vertically incident shear waves, this hypothesis is tested by verifying the sensitivity of the TF idea to different types of waves and incident angles. In the following sections, expressions for the TF parameters and the modification factor are proposed, which constitutes the basis of the method. The paper concludes with some applications followed by conclusions and recommendations for further work.

2 Problem statement

Seismic ground motions in structural systems can be understood like the superposition of waves incident from the source and scattered from the foundation. The scattered component of the field depends upon the kinematic and dynamic properties of the structure-foundation system and these motions are therefore unknown prior to the presence of the building. In this work, a method to address this paradox is proposed by providing the analyst with a spectral description of the seismic excitation, while at the same time considering the scattered field.

The method proceeds like in the classical fixed-base oscillator approach, but yields spectral ordinates that take into account SSI effects. The basis of the method is a spectral response modification factor F_r, which is applied to the traditional 5%-damped design response spectra in order to consider the contribution from the scattered motions. In order to find F_r, a main input parameter is used for the TF between the structural relative displacement $V(i\omega)$ and the free-field motion $V^0(i\omega)$ where i is the imaginary unit and ω represents circular frequency. The TF is described in terms of its two main controlling parameters, namely: its maximum amplitude A_{max} and its associated frequency f_{max}. Two expressions are proposed to find the TF characteristic parameters in terms of standard geometric and dynamic properties of the structure-foundation system as in the classical design approach.

To clarify the proposed technique, R^0 is assumed to be a given spectral response quantity, consistent with free-field conditions. It then follows that the resulting response spectral quantity after considering SSI effects can be obtained according to the relation $R^{SSI}(T_e) = F_r(A_{max})*R^0(T_e/\beta)$, where T_e is the fundamental period of the structure and β is the ratio between f_{max} and the fundamental frequency of the fixed-base structure defined like $\beta = f_{max}T_e$.

Although the proposed method yields results closely related to those of the classical equivalent oscillator technique, there is a conceptual difference between the two methods. Herein the TF is not assumed to exhibit the shape that characterizes a fixed-based oscillator but in contrast, the function reflects the fact that when the SSI effects are strong, the actual TF and that of an equivalent fixed-based SDOF system exhibit substantial differences.

In order to describe the actual TF = $V(i\omega)/V^0(i\omega)$ considering the contribution from the scattered field, a parametric study was conducted for a SDOF system using an in-house numerical code based upon the boundary element method. The model under study assumes a structure characterized by its fundamental period, supported on an infinitely rigid surface foundation, and supported by a homogeneous half-space. The half-space and the structure-foundation system are subjected to vertically incident shear waves of unit amplitude. On the other hand, it is assumed that the structure has a long direction perpendicular to the plane of the analysis and that the vibratory motions occur in the shortest direction (also contained in the plane of the analysis). This implies that the average in-plane response closely matches that of a plane strain idealization and therefore, the structure and the soil, are considered without variations in its mechanical and geometrical properties along the direction perpendicular to the plane of analysis.

The following parameters and corresponding ranges of variation were considered in the parametric analysis. Damping ratio for the half-space ξ_s = [0.02,0.05,0.10], structural slenderness ratio H_e/B = [1.0,2.0,3.0], where H_e is the equivalent height of the single degree of freedom system and B(m) = 32.0 is the foundation width; shear wave propagation velocity for the half-space V_s (m/s) = [200,400,800,1200], structural period T_e(s) = [0.5,1.0,1.5,2.0], mass ratio between the structural and the half-space densities $M_e/(BH_e\rho_s)$ = [0.10,0.20,0.30, 0.40]. To define this last set of values, it was assumed that the average value of the mass per floor in a typical building, considering partition walls and columns, is close to 800 kg/m^2. If on the other hand, it is assumed as average interstory height a value of 3.0 m and an equivalent structural height of 2/3 of the total height, the result is a mean value for the ratio between the structural and soil density equal to 0.25. The Poisson's ratio and mass density of the soil were assumed to remain constant throughout the analysis with values of v_s = 1/3, and ρ_s = 1800 kg/m^3, respectively. Since the impedance functions in the soil depend on frequency and on the ratio B/V_s the values selected for this last relationship are the most important. B/V_s (s) = [0.16,0.08,0.04,0.027] was used so the obtained results are valid for any combination of values of B and V_s within the considered range. The above set of parameters was combined into 576 different SSI-SDOF systems and analyzed with an in-house software based upon the Boundary Element Method (BEM), which effectively takes into account radiation damping.

3 Validation of the numerical framework

Before computing the actual TFs for the parametric analysis, a validation of the BEM code was performed. For that purpose, results from numerical simulations described in terms of the TF, were compared against those obtained using the superposition method described in Clough and Penzien (1993) using the impedance functions derived by Luco and Westman (1972) and corresponding to 2D SSI conditions. The set of parameters reported in Table 1 was used for the validation.

The solutions from both methods are compared in Fig.1, where the results from the numerical simulation are represented by the continuous line, while the discrete diamonds correspond to the response obtained using the impedance functions from Luco and Westman (1972).

4 Transfer functions

Figure 2 shows the TFs corresponding to three extreme cases obtained with the set of parameters reported in Table 2.

Table 1 Parameters for the validation of the numerical framework

T_e (s)	0.6
M_e (kg)	622080
H_e (m)	12
B (m)	32
V_s (m/s)	200
ξ_s	0
ρ_s (kg/m^3)	1800
v_s	0.25

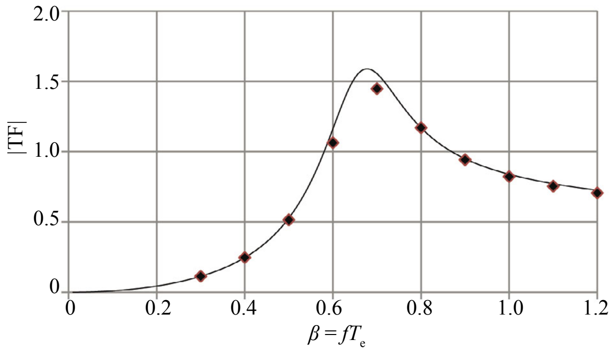

Fig. 1 Comparison between the numerically derived TFs obtained using an in-house boundary element based code (continuous line) and those obtained with a superposition technique using the impedance functions from Luco and Westman (1972) (diamonds)

Case (a) in the top-left part of the figure closely resembles the TF for a fixed-base SDOF system having a well-defined period and a post-peak asymptotic behavior approaching the analytic value of 1.0. Case (b) in the top-right part of the figure shows that the TF exhibits a behavior which is far from the fixed-base SDOF oscillator. This case represents problems where the SSI effect is strong and the post peak response is far from asymptotic to 1.0 in the frequency range of interest. As a final study Case (c) (in the bottom part of the figure) depicts a TF where the interaction is extremely large, with maximum amplitudes for the natural period of the system close to 0.5 and an increasing branch in the low frequency range. This is representative of cases corresponding to heavy structures, with a low value of natural period and located on soft soils. Such cases are out of the domain of application of the current study and only systems having a well-defined bell-shaped TF with a clearly recognizable period and a marked descending branch are considered herein. These two last features allow the TF to be described in terms of the two simple parameters introduced above.

The above observations lead to a more general description of the TF allowing the analysis of systems regardless of the SSI effects being weak or strong. Even though in this approach the TF is characterized in terms

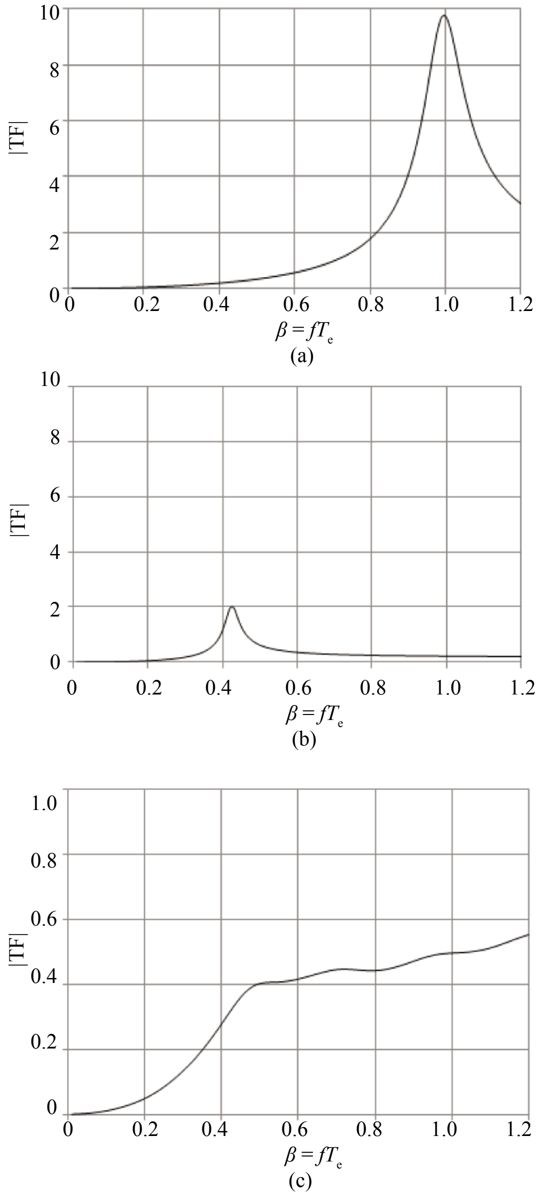

Fig. 2 Amplitudes of the TFs between the relative displacement of the structure and the free field displacement for three extreme cases corresponding to the parametric study conducted in this work

Table 2 Values of the system parameters corresponding to the extreme cases shown in Fig. 2

System parameters	(a)	(b)	(c)
H_e (m)	32	128	12
B (m)	32	32	32
M_e (kg)	92160	1843000	276480
T_e (s)	0.5	3.0	0.2
ξ_e	0.05	0.05	0.05
ρ_s (kg/m^3)	1800	1800	1800
V_s (m/s)	1300	170	200
v_s	1/3	1/3	1/3
ξ_s	0.03	0.03	0.02

of two parameters, a third parameter, corresponding to the asymptotic post peak value of the TF, could also have been considered. It is, however, neglected because its value is strongly correlated with A_{max} and f_{max}.

5 Validation of the hypothesis of vertical incident S-waves

Since in this parametric study assumed vertically incident shear waves, a verification analysis regarding the validity of that condition is performed. For this purpose, the TFs corresponding to P and S waves at different incidence angles and for the model parameters reported in Table 3 were obtained numerically. These parameters were defined in order to trigger kinematic effects, considering that the TF is a relative measure between the relative horizontal structural displacements and the horizontal free-field displacement. In the case of a stiff foundation and non-vertical incidence, the kinematic interaction is different from zero. Thus, not only the horizontal motion of the foundation prior to the interaction is different from the free-field displacements, but there is also an incident rotation of the foundation due to the propagation of a vertical displacement component through the foundation. This last field modifies the relative structural displacements and subsequently the TF. The question that arises is then related to how different is the horizontal foundation displacement with respect to the free-field motion and how large is the incident rotation due to the propagation of the vertical displacement. These motions depend exclusively on the wavelength of the surface displacements and on the foundation width.

For vertical incidence, the horizontal wavelength is infinite and there are no kinematic effects. However, for an incidence angle α, defined with respect to the vertical, the horizontal wavelength is given by $\frac{V}{f\sin\alpha}$, where V is the soil propagation velocity (for a P or S wave) and f is the excitation frequency. For α =15°, V = 400 m/s and f = 2.0 Hz, a wavelength of 772.74 ms results, which compared to the width of a typical foundation implies that the kinematic effects are negligible. Figure 3 shows the TFs for the two types of body waves and for different incidence angles: P waves at 5°, 15°, 30° and 45° and S waves at 0° and 24°. From this plot, it is clear that for values of the parameters within the selected range in this study, the TF exhibits low sensitivity to the type and angle of incident wave.

6 Frequency associated to the maximum amplitude in the TF

Since the frequency associated with the maximum amplitude of the transfer function f_{max}, corresponds to the so-called equivalent frequency of the SSI system, the expression proposed by Avilés and Pérez-Rocha (1998) is used as a starting point for its calculation

Table 3 Model properties used in the verification of the TF approach in the case of non-vertically incident P and S waves

H_e (m)	96
B (m)	32
M_e (kg)	1382400
T_e (s)	2
ξ_e	0.05
ρ_s (kg/m³)	1800
V_s (m/s)	200
ν_s	1/3
ξ_s	0.02

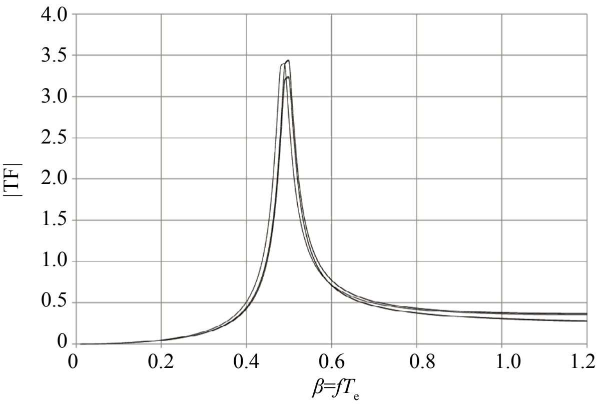

Fig. 3 Transfer functions for different types of waves and incidence angles. P waves were incident at 5°, 15° and 30° while S waves were incident at 0° and 24°

$$\frac{1}{\tilde{\Omega}^2} = \frac{1}{\Omega_e^2} + \frac{1}{\Omega_h^2} + \frac{1}{\Omega_r^2} \tag{1}$$

and where $\tilde{\Omega}$ is the circular frequency of the SSI system, Ω_e the circular frequency of the fixed-base structure and Ω_h and Ω_r are, respectively, the translational and rotational circular frequencies of the structure considered infinitely rigid but supported in a flexible half-space. These last two parameters are defined as:

$$\Omega_h^2 = \frac{K_h}{M_e} \tag{2}$$

$$\Omega_r^2 = \frac{K_r}{M_e H_e^2} \tag{3}$$

where K_h and K_r represent the translational and rotational stiffness of the soil-foundation system evaluated at the frequency $\tilde{\Omega}$. In the case of a 2D half-space, if based on physical considerations, it is assumed that $K_h \propto G_s$ and $K_r \propto G_s B^2$, the shear modulus of the soil is $G_s = V_s^2 \rho_s$, and the structural stiffness is $K_e = 4\pi^2 M_e / T_e^2$. The following expression for the frequency ratio β is found from Eq. (1)

$$\beta = \frac{1}{\sqrt{1+\frac{K_e}{G_s}\left[\alpha_{\beta 1}+\alpha_{\beta 2}\left(\frac{H_e}{B}\right)^2\right]}} \quad (4)$$

In Eq. (4) $\alpha_{\beta 1}$ and $\alpha_{\beta 2}$ are constants, which result from minimization of the quadratic error between the numerical simulations and the estimated computations. From the 576 analyzed cases, $\alpha_{\beta 1}$ = 0.1565 and $\alpha_{\beta 2}$ = 1.8422 is obtained. Figure 4 compares the values for β estimated using expression (4) and those from the numerical simulation. The estimated values have a mean and maximum deviation of 0.0065 and 0.040, respectively. From expression (4), it is clear that for the studied SSI cases, the β values depend only on the ratio between the stiffness of the structure and the shear modulus of the half space and on the slenderness of the structure.

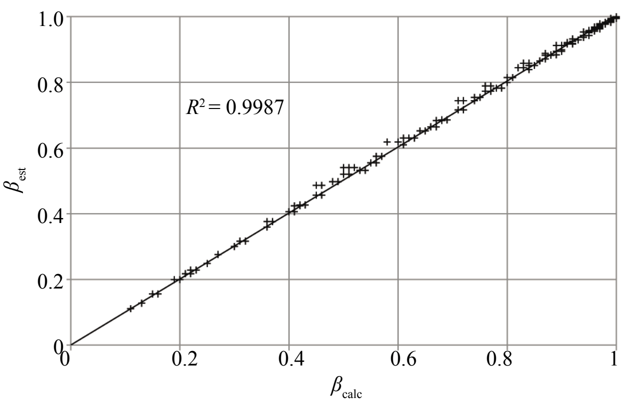

Fig. 4 Comparison between values of the frequency parameter β of the TFs obtained through numerical simulations and those obtained with the expression proposed in Eq. (4)

7 Maximum amplitude

In order to find an expression to compute the maximum amplitude A_{max} of the TF, the expression proposed by Avilés and Pérez Rocha (1998) is used as a starting point to determine the damping ratio in the equivalent fixed-base structure:

$$\tilde{\xi} = \beta^3\left(\xi_e + \xi_h\frac{\Omega_e^3}{\Omega_h^3} + \xi_r\frac{\Omega_e^3}{\Omega_r^3}\right) \quad (5)$$

Considering that $A_{max} = 1/(2\tilde{\xi})$, allows the previously derived relationships to be used after performing some simplifying mathematic manipulations and searching for a good fit to propose the following expression for A_{max}:

$$A_{max} = \frac{1}{2\xi_e\left\{1+\left(\frac{K_e}{G_s}\right)\left[\alpha_{A1}+\alpha_{A2}\xi_s\left(\frac{H_e}{B}\right)^{3/2}\right]\right\}} \quad (6)$$

Once again α_{A1} and α_{A2} are constants resulting from the minimization of the quadratic error between the numerical simulations and the estimated computations. The new set of parameters reads α_{A1} = 2.6649 and α_{A2} = 60.2743. The relation between the estimations using Eq. (6) and the numerical results are displayed in Fig. 5. The mean and maximum found deviation are now 0.217 and 1.573, respectively.

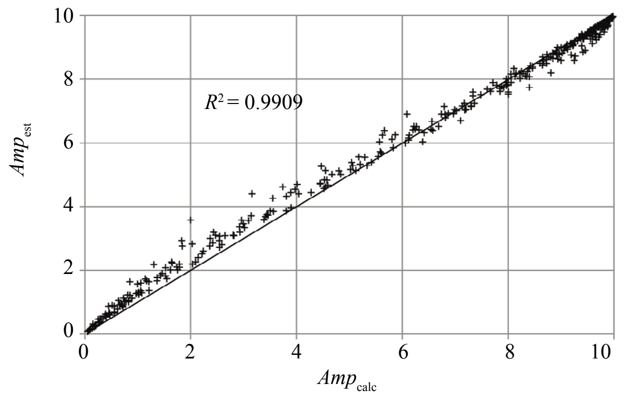

Fig. 5 Comparison between values of the maximum amplitude A_{max} of the TFs obtained through numerical simulations and those obtained with the expression proposed in Eq. (6)

8 Response spectra modification factor F_r

In this section, the idea of a response spectra modification factor F_r is introduced, to relate responses in a structure with and without interaction effects. In particular, F_r relates the maximum displacement for the interacting structure to the maximum displacement of the fixed-base structure but with a fundamental period corresponding to the interacting structure. In order to study the effect of the TFs over actual spectral ordinates, a set of 13 acceleration time histories are considered, corresponding to four ecords in firm soil, four records in soft soil with a fundamental period T_s = 1.0 s, four records in a soil with a fundamental period T_s = 2.0 s and onerecord in soil with a fundamental period T_s = 0.5 s. This is representative of records at the free surface of actual soil deposits. Figure 6 displays the normalized acceleration response spectra for the 13 time histories used as free-field motions.

On the other hand, 10 TFs were selected, and described in terms of their characteristic parameters and covering a wide range of actual structural periods and different levels of interaction. In this set of TFs, a strong interaction level is one that generates considerable changes in the period of the SSI-system with respect to the structural period (e.g., β << 1.0), and considerable changes in the amplification parameter (e.g., A_{max}<< $1/2\xi_e$ which corresponds to the amplification in a structure with no interaction and with a damping ratio of ξ_e). The selected TFs characterized by the parameters defined in Table 4 were later used in the computation of the maximum relative displacement for structures with and without interaction effects The response spectra modification factor defined by the ratio $F_r(A_{max}) = R^{SSI}(T_e)/R^0(T_e/\beta)$, is plotted in Fig. 7 for ξ_{Sa}=0.05 together with the following proposed expression to relate the parameters $F_r v_s A_{max}$ and where ξ_{Sa} corresponds to the free field spectra damping factor.

$$F_r = \left(2\xi_{Sa} A_{max}\right)^{1/3} \tag{7}$$

In order to improve the scatter of the results, an additional correction parameter in the expression for F_r that takes into account characteristic features of the particular signal being used, for instance, the fundamental frequency of the record, is introduced. However, this parameter is difficult to estimate and introduces complexities in the proposed approach. The relationship for the spectral response modification factor F_r, is conservative and captures the fundamental physics of the problem as can be seen from the studied extreme cases. For instance, when TF amplitudes equal $1/(2\xi_{Sa})$, the factor must be equal to 1.0, while it approaches 0.0 as the amplitude approaches 0.0. This is due to the fact that the post-peak asymptotic value in the SSI system decreases with amplitude. This trend, however, is not captured in the TF of a fixed-base system, which is the basis of standard SSI-techniques based on the equivalent oscillator method, and in which case the asymptotic value takes the constant value of 1.0. Due to this last feature of the TF, it is also evident that the slope of the A_{max} and F_r relation is higher for smaller values of the amplitude.

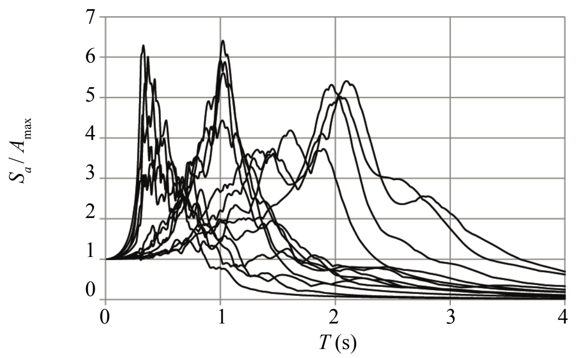

Fig. 6 Normalized acceleration response spectra for the 13 time histories used as free-field motions

Table 4 Parameters of the selected TFs used in the determination of the free-field 5%-damped response spectra modification factor

T_e	β	A_{max}
0.30	0.65	0.93
0.30	0.86	3.05
0.50	0.96	7.16
0.50	0.91	3.42
1.00	0.94	8.18
1.00	0.78	4.29
1.50	0.94	9.30
1.50	0.80	5.77
2.00	0.94	9.83
2.00	0.80	6.36

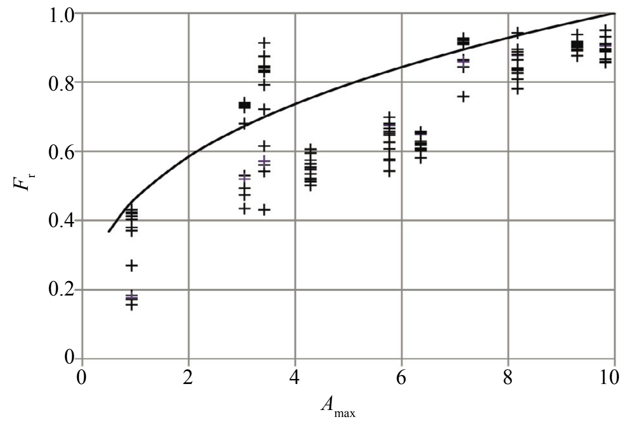

Fig. 7 Relationship between F_r and A_{max} for ξ_{Sa} = 0.05

9 Application

The proposed method is used to compute the response spectra considering interaction effects for several illustrative cases. In each case, the slenderness ratio (H_e/B) of the studied structure is defined and the vibration period, shear wave propagation velocity (V_s), density ratio ($M_e/(BH_e\rho_s)$) and soil damping (ξ_s) are varied. The structural equivalent height H_e, is considered correlated to the structural period according to the expression $T_e = H_e^{0.75}/15$. For the free-field motion, two of the signals considered during the estimation of the response modification factor are used: one of the signals corresponding to firm soil and the second one corresponding to soft soil with fundamental period equal to 1.0 s. Table 5 defines the parameters used in the six study cases.

In each case, the periods for the response spectra shown in Fig. 8 are limited to the periods contained in the domain of applicability of expressions (4) and (6), regardless if this is due to the structural period or to the range of application of the slenderness ratio B/V_s.

From Fig. 8 it is clear that the interaction increases with the slenderness and with the structural mass. It is also evident how in general, for ascending zones of the response spectra considering SSI effects, larger design spectral values are obtained when compared to those without interaction effects. Similarly, for descending zones of the spectra, important reductions are generally obtained for the design values. The role played by the period shift, for which a very close estimation is available with the proposed expression, is highly important in the consideration of SSI effects. This fact can justify not fine tuning the expression to compute the response modification factor F_r, but is at the expense of larger information regarding the shape of the response spectra. The response spectra is difficult to define and its effect in the final result is of little relevance.

Table 5 Properties used in the cases selected to test the current method

	(a)	(b)	(c)	(d)	(e)	(f)
Signal	Rock	Rock	Rock	Soil	Soil	Soil
V_s (m/s)	400	400	400	200	200	200
ρ_s (kg/m^3)	1800	1800	1800	1800	1800	1800
v_s	1/3	1/3	1/3	1/3	1/3	1/3
ξ_s	0.02	0.02	0.02	0.02	0.02	0.02
H_e/B	2.0	2.0	3.0	2.0	2.0	3.0
$M_e/(BH_e\rho_s)$	0.3	0.4	0.3	0.3	0.4	0.3
ξ_e	0.05	0.05	0.05	0.05	0.05	0.05

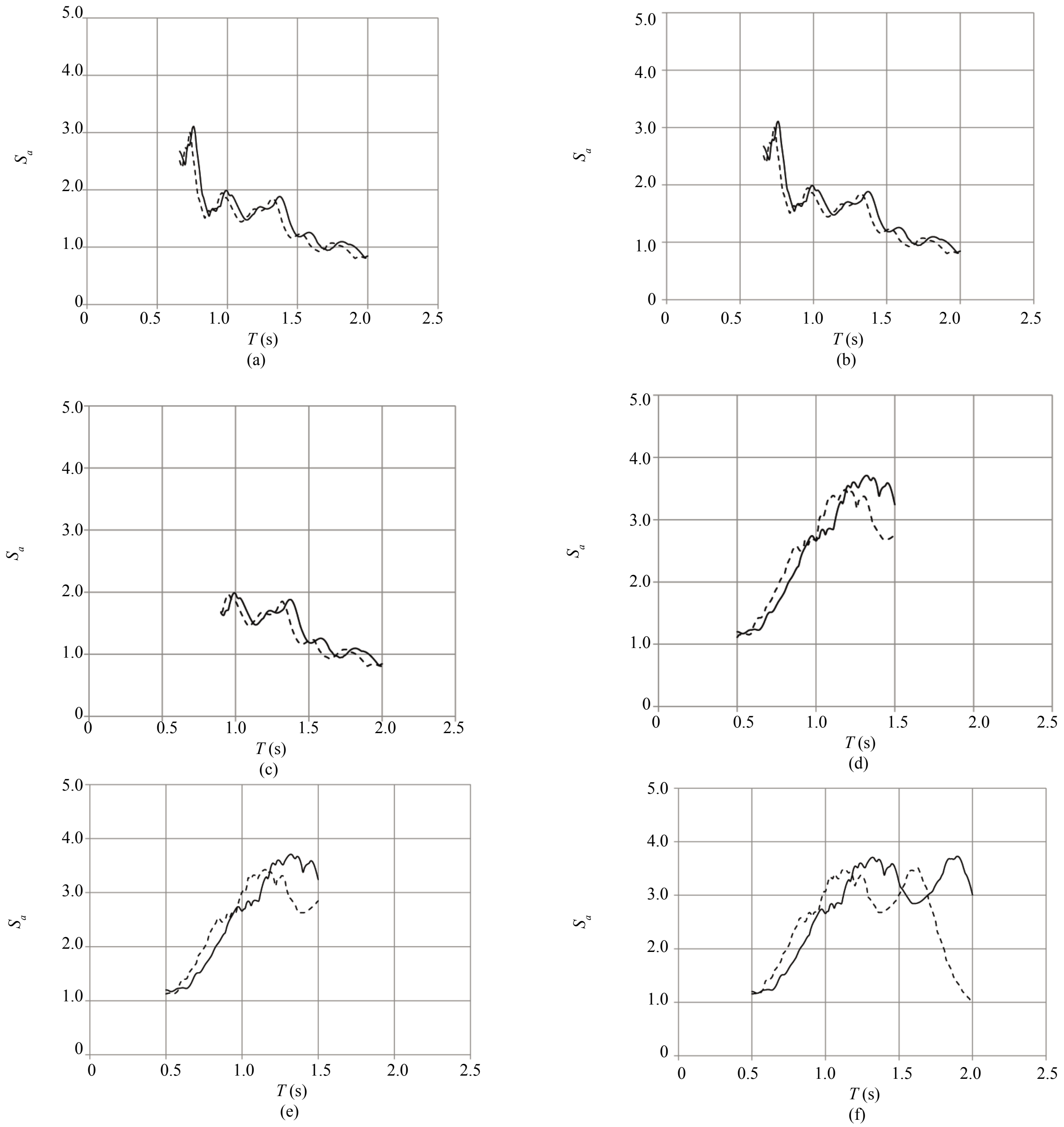

Fig. 8 Response spectra with soil-structure interaction (dashed line) and without interaction (solid line) for the cases defined in Table 5

10 Conclusions and recommendations

An alternative approach to evaluate the spectral earthquake response of 2D-systems, subjected to vertically incident shear waves is described. The method takes into account soil-structure-interaction effects in a simple, yet physically consistent basis. The approach is valid for problems idealized by an infinitely stiff surface foundation, interacting with a homogeneous linear elastic half-space. The technique is attractive since it captures the actual response of a wide range of problems in terms of the same input variables that are currently used in classical seismic regulation techniques.

The method is based upon a description of the transfer function between the structural displacements relative to the base and the horizontal free-field motions. These TFs have been described in terms of its two controlling describing parameters; maximum amplitude A_{max} and its associated frequency f_{max}. Expressions for engineering estimations of both parameters are presented and the resulting values are compared with those obtained through a robust numerical algorithm. The method is completed by an expression to estimate a third parameter, corresponding to a modification factor F_r to be imposed upon the free-field 5%-damped response spectra in order to produce a response spectra modified by soil-structure-interaction effects. The deviations of the first two expressions with respect to the exact responses are very low, resulting in variation coefficients close to 0.01 for the frequency parameter and 0.05 for the amplitude parameter, while higher deviations in the third parameter were observed. This result was expected due to inherent variations in the response spectra associated with different time histories.

The proposed method is still based on classical existing techniques to consider SSI effects and is easily applicable in engineering practice. At the same time, the method spans general cases found in practice and leaves aside neglects extreme cases with associated extreme behaviors that are difficult to formalize in a general strategy. Finally, the results are limited to conditions of linear elastic soil behavior. Under nonlinear conditions however, the impact of the SSI effects should be important due to the loss of stiffness in the soil.

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