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Soft Soil Seismic Design Spectra Including Soil-structure Interaction

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ABSTRACT

Design spectrum is known as an essential tool in earthquake engineering for calculation of maximum (design) responses in a structural system. Soil-structure interaction (SSI), as a phenomenon of coupling of responses of a structure and its underlying soil, was explored after introduction of design spectra and has not been taken into account in developing a design spectrum traditionally. To consider the SSI automatically when doing a spectrum analysis, in this paper maximum response of a single degree of freedom system resting on a flexible base is determined under consistent earthquakes. Consistency of earthquakes is maintained by considering their magnitude, distance, local soil type, and return period. The latter parameter is accounted by the use of earthquake categories identified by their similar spectral values at short periods. Different types of soils and two categories of earthquakes regarding their distance, being near field and far field, are considered. The results are presented as smoothed design spectra. It is shown that SSI alters the response acceleration of buildings having up to about 10 stories and is ineffective for the rest. It has an increasing effect for the response acceleration of buildings up to about 5 stories.

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1. INTRODUCTION

In the design of earthquake resistant buildings, usually using only the maximum responses adequately satisfies the design needs. Spectral analysis of structures had emerged in response to this fact. The design spectrum, as the essential tool for spectral analysis, is a graphical representation of maximum response, usually acceleration, of a single degree of freedom (SDF) system against a suite of earthquake ground motions appropriately averaged and smoothed to fulfill the design needs. The general shape of a response/design spectrum is a graph ascending fast in the range of very short periods, being more or less stable (invariant) at short to relatively medium periods, and descending rapidly to very small values for larger periods. The extent of the region of almost constant spectral accelerations depends primarily on the type of the soil on which the earthquakes behind the spectrum had been recorded, on earthquake magnitudes, and on epicentral distances.

The SDF system used for developing a response or design spectrum is fixed at its base and is identified solely by its natural period. Of course, the damping ratio of such a system is also important but it is taken uniformly as being equal to 0.05 and is not in practice a variable. The SDF system is actually representative of the dynamic characteristics of a specific vibration mode of a multi degree of freedom (MDF) system. After computing the natural frequencies and mode shapes of the MDF system from a characteristic equation, the spectral acceleration is extracted from the design spectrum at each natural period and the maximum responses are calculated from the spectral analysis equations using the dynamic characteristics of the MDF system.

The design spectra have been incorporated in building design codes such as ASCE 7-16 [1].

Nowhere in the above procedure, the soil-structure interaction (SSI) is taken into account. The SSI phenomenon is known by two facts. First, under earthquake waves, the rigidity of foundation of a structure changes the ground motion at the foundation such that it only experiences an average of the ground

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motion corresponding to when no foundation is present, or the free-field motion. This effect appears as six components of input motion, or degrees of freedom, at a three dimensional foundation by displacing and rotating the foundation from its place at rest. It is called the kinematic interaction. The second fact in a SSI problem is that mass of structure and foundation results in a dynamic response in the system hence dynamic reactions at the base of structure on the soil. Looking downward, these reactions by varying with time produce additional waves which make a further change in the free-field motion of soil around the structure. It is called the inertial interaction.

The kinematic interaction has been known to be important for large and rigid foundations while the inertial interaction is essential for massive and rigid/tall structures on relatively soft soils.

Commence of the SSI studies on different structural systems goes back to 1960's. It was developed into a textbook by Wolf [2]. The SSI regulations found their way into the building codes with NEHRP [3] and ASCE 7 [4], but only as an arbitrary design action, where the kinematic interaction has been ignored. Recently, in a more comprehensive approach, ASCE41-13 [5] has presented regulations for both kinematic and inertial interactions and has mentioned the conditions when taking SSI into account is necessary for seismic evaluation of existing buildings.

As the above extensive literature review reveals, correction of the fixed-base response spectra to include SSI into account is untouched effort. This is the incentive and originality of the current work. As such, the purpose of this study is developing seismic design spectra including SSI, such that there will be no need to explicitly model SSI in a structural design problem. By using these spectra, the dynamic characteristics of the fixed-base conventional system are calculated and used in spectrum analysis and the SSI is automatically taken into account.

2. THE SYSTEM UNDER STUDY AND THE EQUATIONS OF MOTION

Figure 1 displays the system for which the dynamic response calculations are to be implemented. It consists of a mass m, damping ζ, and stiffness k, all representatives of structure's characteristics. ζ is taken as 0.05 as usual. Also, the foundation and soil are characterized by their masses m_b and m_v, and their mass moments of inertia, I_b and I_v, respectively, all assumed to be concentrated at the foundation level. The soil springs and dampers, which are in a two-dimensional (2D) view introduced by k_x and c_x in translation and k_ψ and c_ψ in rotation.

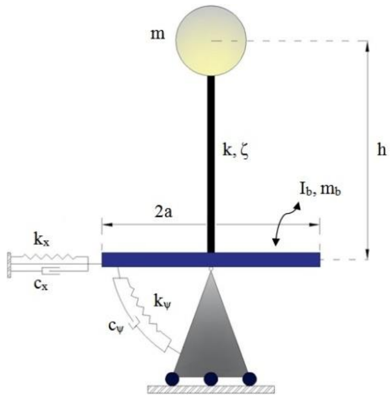


Figure 1. The system under study

The latter quantities simulate the flexibility of soil and radiation of vibration energy from structure toward infinity in soil along with the material damping of soil. While values of the soil spring/damper coefficients vary with the frequency of vibration, it has been shown that the variation is not considerable for homogeneous soils and for low frequencies (e. g., Wolf [2]). Because of this fact, it has been warranted by the building regulations (ASCE 7 [1], NEHRP [3]) to set the spring and damper coefficients at constant values computed at low frequencies considering their governing contribution to structural response.

As mentioned above, the kinematic interaction is negligible for conventional structures. Such an assumption results in a seismic input motion being equal to the free-field ground motion, and is taken in this study too. Then, three equations of motion are written for the three-degree of freedom system of Figure 1 as follows:

Equilibrium of the structural mass in horizontal direction:

$$m\left(\ddot{u}_b+h\ddot{\psi}+\ddot{u}+\ddot{u}_g\right)+c\dot{u}+ku=0 \quad (1)$$

Equilibrium of the total system in horizontal direction:

$$(m+m_b)\ddot{u}_b+mh\ddot{\psi}+m\ddot{u}+(m+m_b)\ddot{u}_g+c_x\dot{u}_b+k_xu_b=0 \quad (2)$$

Equilibrium of the total system in rotation:

$$mh\ddot{u}_b+\left(mh^2+I_b\right)\ddot{\psi}+mh\ddot{u}+mh\ddot{u}_g+c_\psi\dot{\psi}+k_\psi\psi=0 \quad (3)$$

The above equations are written in matrix form as:

$$\begin{bmatrix} m & m & mh \\ m & m+m_b & mh \\ mh & mh & I_b+mh^2 \end{bmatrix}\begin{Bmatrix} \ddot{u} \\ \ddot{u}_b+\ddot{u}_g \\ \ddot{\psi} \end{Bmatrix}+\begin{bmatrix} c & 0 & 0 \\ 0 & c_x & 0 \\ 0 & 0 & c_\psi \end{bmatrix}\begin{Bmatrix} \dot{u} \\ \dot{u}_b \\ \dot{\psi} \end{Bmatrix}+ \begin{bmatrix} k & 0 & 0 \\ 0 & k_x & 0 \\ 0 & 0 & k_\psi \end{bmatrix}\begin{Bmatrix} u \\ u_b \\ \psi \end{Bmatrix}=\begin{Bmatrix} 0 \\ 0 \\ 0 \end{Bmatrix} \quad (4)$$

Equation 4 can be written as follows:

$$[M]\{\ddot{U}\}+[C]\{\dot{U}\}+[K]\{U\}=\{P(t)\} \quad (5)$$

in which:

$$\{U\}=\begin{Bmatrix} u \\ u_b \\ \psi \end{Bmatrix} \tag{6}$$

$$[M]=\begin{bmatrix} m & m & mh \\ m & m+m_b & mh \\ mh & mh & I_b+mh^2 \end{bmatrix}$$

$$m_b=m_f+m_v$$

$$I_b=I_f+I_v \tag{7}$$

$$[C]=\begin{bmatrix} c & 0 & 0 \\ 0 & c_x & 0 \\ 0 & 0 & c_\psi \end{bmatrix} \tag{8}$$

$$[K]=\begin{bmatrix} k & 0 & 0 \\ 0 & k_x & 0 \\ 0 & 0 & k_\psi \end{bmatrix} \tag{9}$$

$$\{P(t)\}=-\begin{Bmatrix} m \\ m+m_b \\ mh \end{Bmatrix}\ddot{u}_g(t) \tag{10}$$

In the above equations, u is the horizontal displacement of the mass m with respect to an axis perpendicular to the foundation's plan, ub is the added horizontal displacement of foundation with respect to the free field ground motion, ug is the free-field ground motion with respect to a fixed reference, ψ is the rotation angle of motion of foundation, and a dot shows differentiation with respect to time.

Equation (5) will be solved, using the Newmark constant acceleration numerical integration procedure [9] to calculate the total acceleration, $\ddot{u}+h\ddot{\psi}+\ddot{u}_b+\ddot{u}_g$ of the mass m and its maximum, S_a, under each earthquake. Before being able to implement such a calculation, values of the soil spring/damper coefficients, or impedances, and mass (inertia) in the left-side of Equation (5) and the suitable earthquake acceleration time histories, at the right-side of Equation (5) must be determined. These issues are described in the following sections.

3. THE SOIL IMPEDANCES AND THE NON-DIMENSIONAL EQUATIONS

A soil-structure dynamic system can be modeled taking the soil and structure parts concurrently in a single dynamical modal, or separately. The former is called the direct method, that is too large with a SDF structure on soil, and the latter is known as the substructure method. In the substructure method the soil impedances are calculated separately and used in a model containing only the structure and foundation. For a 2D SSI problem of an SDF structure, the substructure method results in a model, exactly as appears in Figure 1, for calculating the structural response.

There have been many studies on computation of soil impedances for use in the substructure method. Many of these works resulted in graphs and formulas for calculation of frequency dependent impedances. As mentioned above, it is usually warranted to use frequency independent impedances as a good approximation in the period range important in earthquake engineering. The code-based formulas of impedances, such as ASCE 41-13 [4], are only given for springs and use of an equivalent damping for the structure is allowed. There have been also published equations for both frequency-independent spring and damper impedances, such as Mulliken and Karabalis [5].

Due to its completeness and simplicity, in this study the equations proposed in Mulliken and Karabalis [5] for soil's impedances and masses are used. These equations are mentioned in Table 1 with modification for the degrees of freedom of the system of Figure 1.

In Table 1, a is half dimension of a rigid equivalent rectangular foundation, and υ, ρ, G, and Vs are the Poisson's ratio, mass density, shear modulus, and shear wave velocity of the soil in the vicinity of foundation.

Applying the values of Table 1 in the system of Figure 1 represents a soil-structure system tuned to the fundamental mode of vibration. Then, using Table 1, Equation (5) can be written in nondimensional form as: in which:

$$\rho\bar{m}\frac{\bar{s}^3\times V_s^3}{\omega^3\times\bar{h}^2}\begin{bmatrix} 1 & 1 & \frac{\bar{s}\times V_s}{\omega} \\ 1 & (1+\bar{m}_b)+\frac{1.0918}{2-\nu}\times\frac{1}{\bar{h}\times\bar{m}} & \frac{\bar{s}\times V_s}{\omega} \\ \frac{\bar{s}\times V_s}{\omega} & \frac{\bar{s}\times V_s}{\omega} & \left(\frac{\bar{s}^2\times V_s^2}{\omega^2}+\frac{2}{3}\frac{\bar{s}^2\times V_s^2}{\omega^2\times\bar{h}^2}(1+\bar{m}_b)+\frac{1.24}{1-\upsilon}\times\frac{\bar{s}^2\times V_s^2}{\bar{m}\times\omega^2\times\bar{h}^3}\right) \end{bmatrix}\begin{Bmatrix} \ddot{u} \\ \ddot{u}_b \\ \ddot{\psi} \end{Bmatrix}+$$
$$\begin{bmatrix} 2\zeta\times\rho\bar{m}\frac{\bar{s}^3\times V_s^3}{\omega^2\times\bar{h}^2} & 0 & 0 \\ 0 & \frac{1.5\rho}{2-\upsilon}\times\frac{\bar{s}^2\times V_s^3}{\omega^2\times\bar{h}^2} & 0 \\ 0 & 0 & \frac{2.4\rho}{1-\upsilon}\times\frac{\bar{s}^4\times V_s^5}{\omega^4\times\bar{h}^4} \end{bmatrix}\begin{Bmatrix} \dot{u} \\ \dot{u}_b \\ \dot{\psi} \end{Bmatrix}+\begin{bmatrix} \rho\bar{m}\frac{\bar{s}^3\times V_s^3}{\omega\times\bar{h}^2} & 0 & 0 \\ 0 & \frac{9.2\rho}{2-\upsilon}\times\frac{\bar{s}\times V_s^3}{\omega\times\bar{h}} & 0 \\ 0 & 0 & \frac{4\rho}{1-\upsilon}\times\frac{\bar{s}^3\times V_s^5}{\omega^3\times\bar{h}^3} \end{bmatrix}\begin{Bmatrix} u \\ u_b \\ \psi \end{Bmatrix}=$$
$$-\rho\bar{m}\frac{\bar{s}^3\times V_s^3}{\omega^3\times\bar{h}^2}\begin{Bmatrix} 1 \\ (1+\bar{m}_b)+\frac{1.0918}{2-\nu}\times\frac{1}{\bar{h}\times\bar{m}} \\ \frac{\bar{s}\times V_s}{\omega} \end{Bmatrix}\ddot{u}_g(t) \tag{11}$$

TABLE 1. Frequency independent soil impedances and masses (inertia)

Degree of freedom	mass (inertia), m_v (I_v)	Stiffness, k	Damping, c
Horizontal	$\frac{4.368(1-\upsilon)\rho a^3}{7-8\upsilon}$	$\frac{9.2Ga}{2-\upsilon}$	$\frac{1.5\rho a^2 V_s}{2-\upsilon}$
Rocking	$\frac{3.403\rho a^5}{1-\upsilon}$	$\frac{4.0Ga^3}{1-\upsilon}$	$\frac{2.4\rho a^4 V_s}{1-\upsilon}$

$$\bar{h}=\frac{h}{a} \tag{12}$$

$$\bar{s}=\frac{\omega_f h}{V_s} \tag{13}$$

$$\bar{m}=\frac{m}{\rho a^2 h},\ \bar{m}_b=\frac{m_b}{m} \tag{14}$$

In the above equations, ωf is the rigid-base natural frequency of system, $\omega_f=\sqrt{k/m}$, $\bar{h}$ is the aspect ratio of structure, $\bar{s}$ is the stiffness to height ratio of structure with regard to soil, and $\bar{m}$ is the structure to soil mass ratio.

The parameters involved in Equations (11)-(14) must be quantified before spectrum analysis. To cover the whole practical range, the natural period of the system of Figure 1 when fixed at based, $T_f=2\pi/\omega_f\approx 0.1n$, corresponding to the fundamental mode of an n-story building, is taken to be 0-4 sec with 0.02 sec increments (totally 200 SDF systems). The damping ratio of such a system is set at 0.05 as usual. Mass of the foundation, mb, is assumed to be equal to the average mass of each story of the n-story building, or $m_b=m/n$. For the nondimensional parameters of Equations (12)-(14) corresponding to the conventional buildings, certain values have been reported in the literature, e.g. [2]. Ranges of 0.5-5 for $\bar{h}$, corresponding to squat to slender buildings, 0.5-5 for $\bar{m}$, corresponding to light to massive structures with regard to soil, and 0.1-10 for $\bar{s}$, corresponding to soft/short to stiff/tall structures with respect to soil, have been recommended. In this study, to find more practical values for the above parameters, a survey on buildings having 1-35 stories is done and the required data is gathered. The results of parameter estimation are given in Table 2 for the selected buildings.

The above values will be used for solving Equation (11) and for the spectrum calculations. However, the final outcome will not be much sensitive to the certain values of the above parameters because the envelope of the resulted spectra will be used to produce a generic smoothed design spectrum.

TABLE 2. Representative parameters of buildings

	Physical Properties								Normalized Properties		
Site Class	**Building Category**	**Number of Stories**	**Foundation Width**	**Height**	**Structure's Mass**	**Soil Density**	**Structure's Period**	**Shear wave velocity**	**Normalized Height**	**Normalized Mass**	**Stiffness Ratio**
D	Case 1	2	16	8	640	1.75	0.25	350	0.5	0.09	0.57
	Case 2	7	10	8	875	1.75	0.6	00	2.8	0.5	1.47
	Case 3	12	25	8	9375	1.75	1.2	50	1.92	0.34	1.01
	Case 4	35	28	40	34300	1.75	4	50	5	0.89	0.88
E	Case 1	2	16	8	640	1.4	0.2	50	0.5	0.11	1.68
	Case 2	7	12.5	8	1367	1.4	0.6	20	2.24	0.5	2.44
	Case 3	12	20	8	13500	1.4	2	50	1.6	0.36	1.01
	Case 4	35	38	40	34300	1.4	4	50	5	1.12	1.47

4. THE GROUND MOTION SELECTION AND MODIFICATION

For computation of design spectra, a number of earthquake records, as large as possible, should be used to have a minimum scatter of results. There are also methods available for record selection that keep the number of records small enough for a certain reliability, such as Baker [6], but they are not used here to retain simplicity.

The PEER strong motion database [7] is consulted for record selection. The earthquake records are sorted out of the database based on their epicenteral distance and the soil on which they were recorded. Since the SSI is more highlighted on softer soils, only soil types D and E are considered for the rest of analysis. Soil types D & E are representative of soft and very soft soils and are introduced in ASCE 7-16 [1]. Their characteristics are selected to be as mentioned in Table 3.

As number of the records on soil type E is not large enough in the database, records with PGA>0.2g on soil D and with PGA>0.05g on soil E are extracted from the database. The earthquake records are divided in two groups based on their epicentral distances as near field with R<20km and far field with R≥ 20km.

The above criteria results in the number of earthquakes being equal to what is mentioned in Table 4, with a total of 476 earthquakes (as of April, 2013).

TABLE 3. Characteristics of the soil types considered

Soil type	V_s (m/s)	υ	$\gamma(kg/m^3)$
D	200	0.4	1800
E	100	0.45	1700

TABLE 4. Number of selected earthquake records

Soil type	Near field	Far field	Total
D	231	60	291
E	37	148	185

Obviously, the above sorted dataset of records contains earthquakes with very different probabilistic properties, i.e., with various return periods. To harmonize the dataset regarding the above fact, the response spectrum of an SDF system, i.e., the system of Fig. 1 fixed at its base, is computed under each of the earthquakes of Table 4. The spectral shape of a response spectrum in short to medium periods is represented by a parameter known as SDS in ASCE 7-16 [1]. SDS is the value of the spectral acceleration averaged in the period range of 0.1-0.5 sec.

Values of SDS representative of a seismic region are 0.5-1.5g in the U.S., with g being the acceleration of gravity. They are given for the U.S. as isoseismic maps by USGS[2] and are included in ASCE 7-16 [1], for two return periods of 475 and 2475 years, where the former and latter are associated with the design and the maximum expectable earthquakes, respectively.

The earthquakes of Table 4 are categorized based on their SDS. The range of SDS in each category should be selected as small as possible. The ranges 0.4-0.6, 0.6-0.8, 0.8-1.0, 1.0-1.2, 1.2-1.4, and 1.4-1.6g are deemed to be appropriate, as described below. The earthquakes of each interval are scaled such that their SDS is equal to the value corresponding to the center of that interval, i.e., 0.5, 0.7, 0.9, 1.1, 1.3, and 1.5g. Obviously, by selecting the SDS ranges as above, the scale factors will be very near to unity and the records are not varied too much. Number of earthquakes in each interval should not be too small for spectrum analysis. Dynamic analysis with at least seven earthquakes is known to be reliable enough to justify use of the average of results [1]. Then, if in an interval described above, there is less than seven earthquakes available, an enough number of earthquake records of the neighboring intervals, those nearer to the boundary, are scaled up or down, as necessary, to be grouped in the needing interval. The scale factor is restricted to 0.5-2 for a minimum change of the original record [8].

The number of records, calculated as above and distributed between the S_{DS} intervals, is as mentioned in Tables 5 and 6.

[2] http://earthquake.usgs.gov/designmaps/

TABLE 5. Number of (scaled) records on soil type D

S_{DS}(g)	0.4≤ S_{DS}<0.6	0.6≤ S_{DS}<0.8	0.8≤ S_{DS}<1.0	1≤ S_{DS}<1.2	1.2≤ S_{DS}<1.4	1.4≤ S_{DS}≤1.6
Near field	15	15	15	15	15	14
Far field	15	15	15	15	9	7

TABLE 6. Number of (scaled) records on soil type E

S_{DS}(g)	0.4≤ S_{DS}<0.6	0.6≤ S_{DS}<0.8	0.8≤ S_{DS}<1.0	1≤ S_{DS}<1.2	1.2≤ S_{DS}<1.4	1.4≤ S_{DS}≤1.6
Near field	15	15	13	9	9	8
Far field	15	8	7	7	7	7

5. CALCULATION OF THE SPECTRA

In this section, by using Equation (11) along with the non-dimensional parameters and the earthquakes introduced above, the maximum total acceleration of the mass m in Figure 1 including SSI, averaged between different earthquakes, is calculated for each case. The peak accelerations are depicted versus the fixed-base period of the mass m, T_f, and presented as both un-smoothed and smoothed design spectra. Moreover, the design spectra are given for the system of Figure 1 fixed at base, i.e. an SDF system, in each case for comparison. The design spectra introduced by ASCE7-10 for the soil types D and E with spectral parameters consistent with each interval of SDS are also presented as a reference in figures.

It should be noted that in addition to the soil type, the SDS and SD1 values are needed for calculation of an ASCE7-10 design spectrum. SD1 is the spectrum value at a period of one second. The following procedure is followed in this study to calculate SD1.

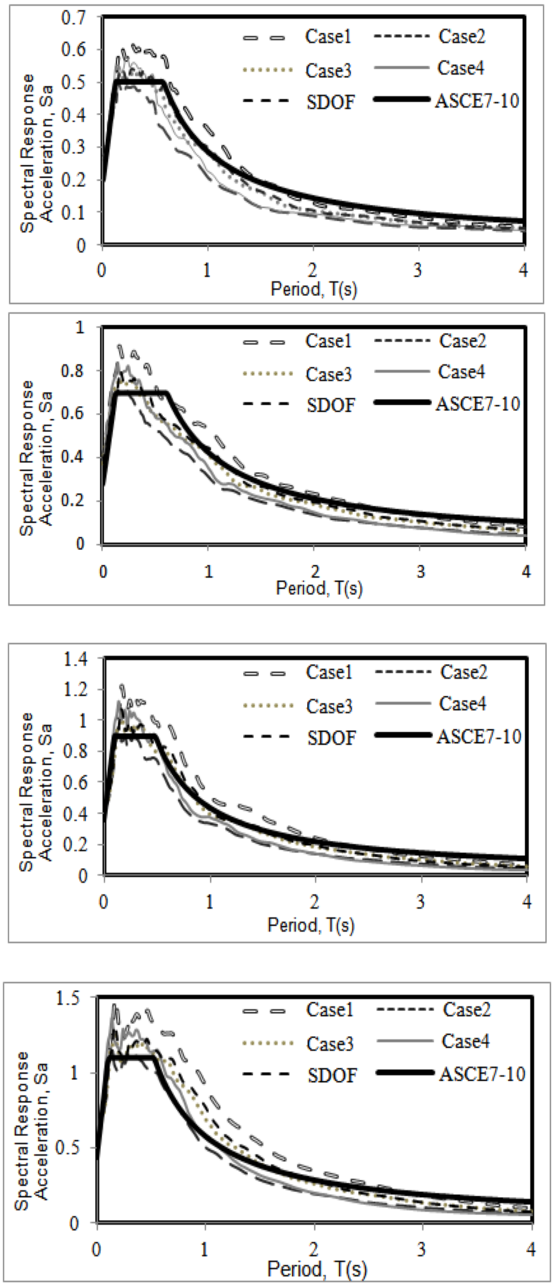
First S_S, the short period spectral amplitude on the bed rock, is calculated using S_{DS} and the soil type characteristics. Utilizing maps of ASCE7-10 for Ss, points with the calculated Ss are located. Then, using the maps of S_1, the S_1 values for the same points are extracted. A single S_1 is calculated by a geometric averaging. Finally, using the soil type characteristics, the S_{D1} values are determined. Table 7 shows the values of S_{D1}, calculated as above, for SDS values used in this study.

TABLE 7. The S_{DS} and S_{D1} values for the ASCE7-10 design spectrum

S_{DS}		0.5	0.7	0.9	1.1	1.3	1.5
S_{D1}	**Soil D**	0.285	0.425	0.44	0.58	0.65	0.75
	Soil E	0.36	0.56	0.69	0.87	0.98	1.1

Finally, since the design spectra are meant to include all of the possible earthquakes in an area regarding their epicenteral distances, only the envelope of the near and far-field earthquakes is presented in the following.

The spectra are labeled regarding building categories of Table 2 as cases 1-4 and presented along with the fixed-base and ASCE7-10 design spectra, in Figures 2 and 3.



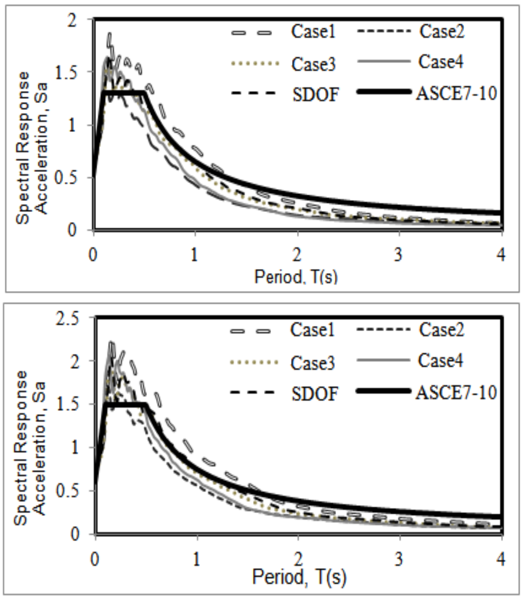
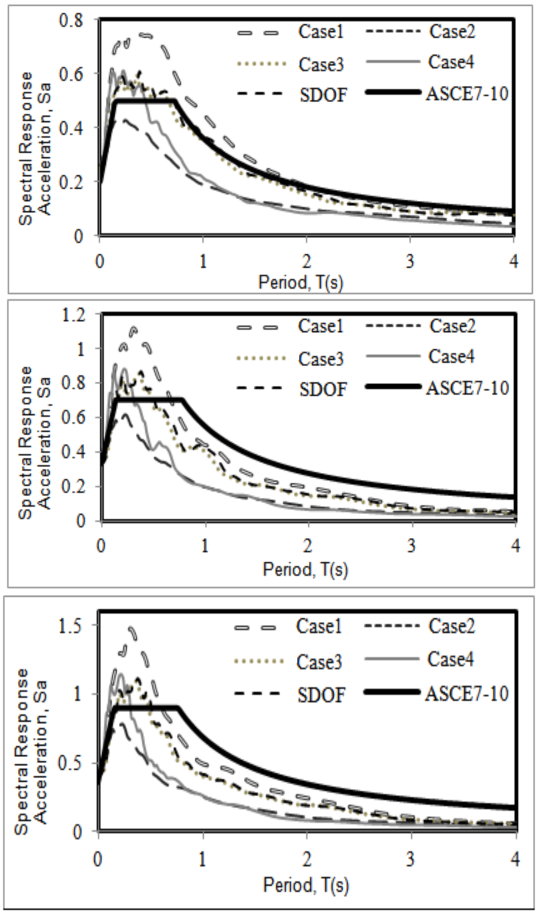


Figure 2. Design spectra for the soil type D with S_{DS}=0.5-1.5



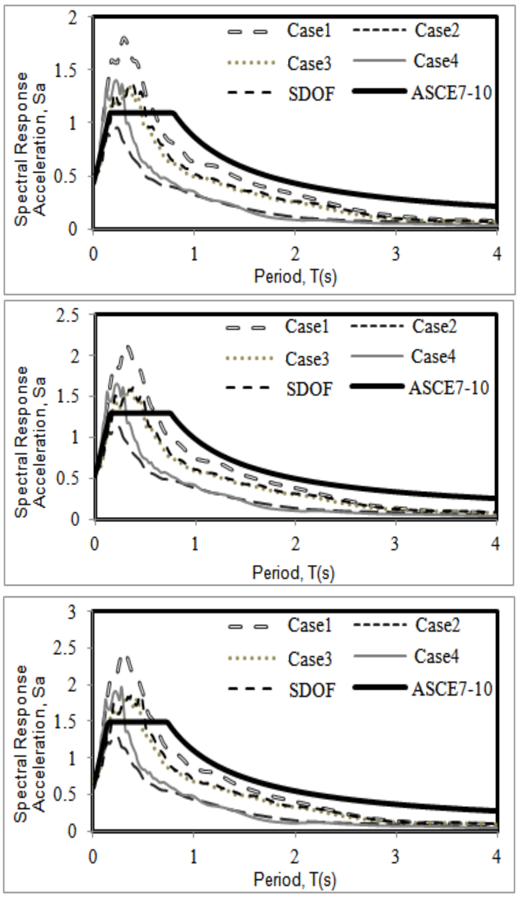


Figure 3. Design spectra for the soil type E with S_{DS}=0.5-1.5

In the above figures, the SDF (no SSI) spectra are in all of the cases above the code-based spectra in the short period range and, in most cases, lower than that for the rest of periods. The difference is larger for the soil type E where the SDF spectra are above and below the code spectra on the average at about 23 % and 68%, respectively.

Moreover, it is observed that in Cases 2-4, the SSI spectra differ from the fixed-base spectrum mostly in the period range of T≤0.75sec, and in Case 1, up to a period of about 2.5sec. Thus, the same period ranges show the interval where the SSI effects are important in each case. Note that according to Table 2 and the description above it, more or less, Case 1 corresponds to squat-light-soft (or short) structures, Case 2 to slender-massive-stiff (or tall) structures, Case 3 to squat-light-stiff (or tall) structures, and Case 4 to slender-massive-stiff (or tall) structures. In the mentioned period range, SSI spectrum of Case 1 is always above that of the fixed-base case. On the other hand, it should be noted that each SSI spectrum corresponds to a certain building category in Table 2, with its mentioned fundamental period. If it is assumed that the dynamic response of a building is sensitive only to periods in the range of 0.2T1-1.5 T1 where T1 is the fundamental period, then it can be said that based on Figures 2 and 3, within 1-35 story buildings, SSI alters the response acceleration of buildings having up to about 10 stories and is ineffective for the rest. It has an increasing effect for the response acceleration of buildings up to about 5 stories.

It is very important to remember that based on what is mentioned following Table 1, the above discussion only applies to the fundamental mode of vibration. It is assumed that SSI does not change the spectral ordinates pertaining to higher modes.

6. THE GENERIC SSI DESIGN SPECTRUM

Upon looking into Figures 2 and 3 and its following discussion, a generic design spectrum including SSI can be derived for the fundamental mode. For this purpose, the part of each SSI spectrum located between 0.2T1-1.5T1 is extracted on each soil type for each SDS. An envelope spectrum is drawn on the above parts. This is shown in Figure 4 along with the corresponding ASCE7-10 design spectrum, e. g., for SDS=0.9.

Because of similarity of graphs, the equation of the generic SSI spectrum is adopted to be similar to that of ASCE7-10 with modification factors as follows:

$$\begin{cases} (S_a)_{SSI}=a_1 S_{DS}(0.4+0.6\,T/T_0) & :T<T_0 \\ (S_a)_{SSI}=a_1 S_{DS} & :T_0\le T\le bT_s \\ (S_a)_{SSI}=a_2 (S_{D1}/T) & :bT_s\le T\le T_L \\ (S_a)_{SSI}=S_{D1}\,T_L/T^2 & :T>T_L \end{cases} \quad (15)$$

in which, (Sa)SSI is the spectral acceleration of the fundamental mode including SSI and a1, a2, and b are modification factors dependent on the soil type and SDS, which are given in Table 8.

TABLE 8. Values of the modification factors for converting the ASCE7-10 spectrum to the SSI spectrum. (a) Soil type D, (b) soil type E. Note: The values are for the fundamental mode. They are equal to unity for the higher modes

(a)

S_{DS}	a_1	a_2	b
0.5	1.240	1.000	0.807
0.7	1.310	0.996	0.725
0.9	1.350	1.110	0.822
1.1	1.360	1.290	0.948
1.3	1.460	0.968	0.662
1.5	1.530	1.020	0.662

(b)

S_{DS}	a_1	a_2	b
0.5	1.5000	1.000	0.667
0.7	1.6000	0.720	0.450
0.9	1.640	0.526	0.417
1.1	1.680	0.638	0.380
1.3	1.7000	0.676	0.400
1.5	1.7000	0.742	0.436

7. CONCLUSION

In this paper, generic design spectra were derived for soft and very soft soils based on the acceleration spectral amplitude in the short period range.

For this purpose, a single degree of freedom system, representative of the modal characteristics of a multistory building, resting on translational and rotational springs and dampers was considered. The non-dimensional equations of motion of such a system were solved using a numerical integration scheme. The non-dimensional parameters were selected to be corresponding to actual building cases. For selection of consistent ground motions, PGA, epicentral distance, and the soil type were taken into account and about 480 earthquake records were adopted.

The earthquakes were sorted based on their SDS to provide for their consistency when computing the response spectra. Based on the results a generic SSI spectrum was presented following equations similar to those of the code-based spectrum but with additional modification factors. Values of the modification factors were presented in tables depending on the soil type and values of SDS.

Using the presented SSI spectra, spectrum analysis of structures can be performed still on the fixed-based models of multi-story buildings and the soil-structure interaction is automatically taken into account.

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Soft Soil Seismic Design Spectra Including Soil-structure Interaction

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چکیده

طیف طراحی به عنوان یک ابزار ضروری در مهندسی زلزله برای محاسبه حداکثر (طراحی) پاسخ در یک سیستم سازه ای شناخته شده است. اندرکنش خاک و سازه (SSI)، به عنوان یک پدیده ترکیب واکنش های یک سازه و خاک زیر آن، پس از معرفی طیف های طراحی مورد بررسی قرار گرفت و در طراحی یک طیف طراحی به طور سنتی مورد توجه قرار نگرفت. برای در نظر گرفتن SSI به طور خودکار هنگام انجام یک تحلیل طیفی، در این مقاله، حداکثر پاسخ یک سیستم یک درجه آزادی که بر روی یک پی منعطف قرار دارد تحت زمین لرزه های همسان تعیین می شود. همسان کردن زمین لرزه ها با توجه به اندازه، فاصله، نوع خاک محلی و دوره بازگشت آنها تعیین می شود. پارامتر دیگر استفاده از زمین لرزه های طبقه بندی شده با مقادیر شتاب طیفی کوتاه مشابه می باشد.در این مطالعه انواع مختلف خاک و میدان دور و میدان نزدیک نیز در نظر گرفته شده است. نتایج به صورت طیف طراحی صاف ارائه می شوند.طیف های طراحی محاسبه شده نشان می دهد که SSI شتاب واکنش ساختمان هایی را که بیش ۱۰ طبقه دارند تغییر می دهد و برای بقیه ناکارآمد است. اندرکنش خاک و سازه اثر فزاینده ای را برای شتاب واکنش ساختمان های بیش از ۵ طبقه دارد.

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