CORRECTION FACTORS INCLUDING NONCLASSICAL NATURE OF SOIL-STRUCTURE INTERACTION SPECTRAL ANALYSIS

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ABSTRACT

The problem of non-classical dynamic analysis of structures resting on flexible bases is studied in this paper. Because of presence of the underlying soil in the dynamic model of structure that acts like an energy sink, the damping matrix is not proportional to structural mass and stiffness and theoretically a non-classical approach should be followed in modal analysis. Considering one to twenty-story buildings, two types of soils, and several suits of ground motions each containing 10 earthquake records specifically selected for each building, the seismic responses are calculated using a time history modal analysis in this paper. Three cases are considered: fixed-base buildings with classical analysis, flexible-base buildings with classical and non-classical analysis. Using the nonclassical analysis, it is shown that soil-structure interaction should not be taken into account for moment frame buildings with the fundamental fixed-base periods smaller than 1 second. Cases for which the base flexibility should be considered for the higher modes too are distinguished. Finally, it is made clear that on each soil type, when the actual non-classicalnature of the SSI system must be accounted for.

INTRODUCTION

Unlike fixed-base systems in which the source of vibration damping is more or less uniquely attributed to the structural system and is hence almost uniform, in a soil-structure-interaction (SSI) system a considerable part of damping is contributed by the totally different medium of soil. The damping matrix of such a complex system is a non-uniform combination of structure and soil damping values and therefore is not classical.

This is while in daily spectral analysis of structures the damping matrix is always presumed to be classical, i.e., proportional to mass and stiffness matrices. When there is a doubt on validity of this basic assumption, likein SSI problems as discussed above, availability of a spectrum analysis methodology corrected for nonclassical damping while retainingits simplicitywill be very helpful.

The work of Veletsos& Venturawas an important step forward in this regard. They simplified the nonclassical modal analysis through giving insight to the physical meaning of differentterms of the formulation and converted the complex-valued equations to their real counterparts. They derived equations for determining natural periods and mode shapes of nonclassical systems resulting in free vibration responses and a Duhamel integral formulation for computing the dynamic response (Veletsosand Ventura, 1986).

Ziaeifar and Tavousi developed formulas for calculating the modal values of the response maxima based on the work of Veletsos& Ventura (Ziaeifar and Tavousi, 2005).

Zhou&Yu derived formulas for combining the maximum modal responses of non-classically damped linear systems. They used the random vibration theory and accounted for the correlation between the modal displacement and velocity responses of structure (Zhou & Yu, 2008). Based on a general modal response history analysis formulation for nonclassical and over-damped systems developed by Song et al. they derived a response spectrum analysis approach and proposed a general modal combination rule (Song et al., 2008).

To gain attractiveness in practical earthquake engineering, a nonclassical SSI problem should be solved within the frame work of a conventional design spectrum.

In the current study, the periods from which SSI should be taken into account are identified. Then it is made clear when the higher mode SSI effects are important and finally, cases for which non-classical SSI analysis are necessary are recognized. One to 20-story buildings are considered and for each building, two suits of ground motion each containing 10 records, one being recorded at the near field and the other at the far field, selected through a special procedure specifically for each structure are used. Two types of medium and soft soils are also considered.

MODAL ANALYSIS OF SOIL-STRUCTURE SYSTEMS

EQUATIONS OF MOTION

A multistory structure on flexible soil subjected to a horizontal ground motion is shown in Fig1.

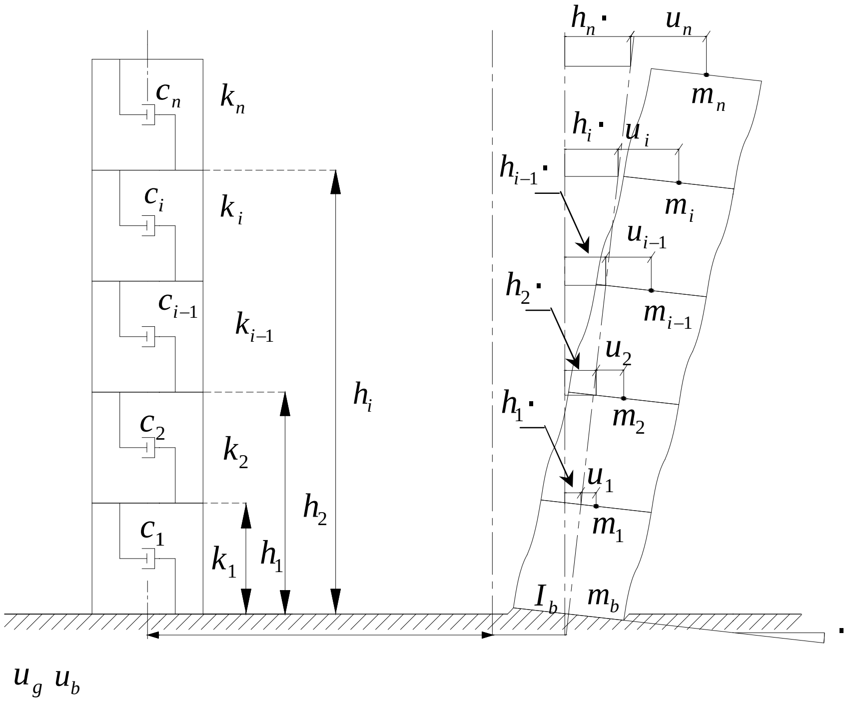

Figure 1. The multistory structure on flexible soil under lateral movement

The system is assumed to be a shear building having a single horizontal degree of freedom (DOF) at each floor to retain simplicity. In addition, it is supposed that the supporting medium possesses a horizontal as well as a rotational DOF, and the input motion in the presence of structure is assumed to be identical to the free-field motion. The equations of motion of the system of Fig.1 can be written as follows:

$$[M]\begin{Bmatrix}\{\ddot{u}\}\\ \ddot{u}_b\\ \ddot{\theta}\end{Bmatrix}+[C]\begin{Bmatrix}\{\dot{u}\}\\ \dot{u}_b\\ \dot{\theta}\end{Bmatrix}+[K]\begin{Bmatrix}\{u\}\\ u_b\\ \theta\end{Bmatrix}=-\begin{Bmatrix}\{m\}\\ m_b+\sum_1^n m_i\\ \sum_1^n m_i h_i\end{Bmatrix}\ddot{u}_g(t)=\{p(t)\} \tag{1}$$

in which:

$$[M]=\begin{bmatrix} [m] & \{m\} & \{mh\} \\ \{m\}^T & m_b+\sum_1^n m_i & \sum_1^n m_i h_i \\ \{mh\}^T & \sum_1^n m_i h_i & I+\sum_1^n m_i h_i^2 \end{bmatrix} \tag{2}$$

$$[C]=\begin{bmatrix} [c] & \{0\} & \{0\} \\ \{0\}^T & c_{uu} & 0 \\ \{0\}^T & 0 & c_{..} \end{bmatrix}$$

$$[K]=\begin{bmatrix} [k] & \{0\} & \{0\} \\ \{0\}^T & k_{uu} & 0 \\ \{0\}^T & 0 & k_{..} \end{bmatrix}$$

and:

$$[m]=\begin{bmatrix} m_1 & & & & \\ & m_2 & & 0 & \\ & & \ddots & & \\ & 0 & & m_{n-1} & \\ & & & & m_n \end{bmatrix}$$

$$[C]=\begin{bmatrix} c_1 & -c_2 & \cdots & 0 & 0 \\ -c_2 & c_1+c_2 & -c_3 & 0 & 0 \\ \vdots & \vdots & \ddots & \vdots & \vdots \\ 0 & 0 & \cdots & c_{n-1}+c_{n-2} & -c_n \\ 0 & 0 & \cdots & -c_n & c_n+c_{n-1} \end{bmatrix} \tag{3}$$

$$[K]=\begin{bmatrix} k_1+k_2 & -k_2 & \cdots & 0 & 0 \\ -k_2 & k_2+k_3 & -k3 & 0 & 0 \\ \vdots & -k_3 & \ddots & \vdots & \vdots \\ 0 & \vdots & \cdots & k_n+k_{n+1} & -k_{n-1} \\ 0 & 0 & \cdots & -k_{n-1} & k_n \end{bmatrix}$$

and:

$$\begin{gathered} \{m\}=[m_1 \quad m_2 \quad \cdots \quad m_n]^T \\ \{mh\}=[m_1h_1 \quad m_2h_2 \quad \dots \quad m_nh_n]^T \\ \{u\}=[u_1 \quad u_2 \quad \cdots \quad u_n]^T \\ I=I_b+\sum_{i=1}^{n} I_i \end{gathered} \tag{4}$$

Parameters of the above equations should be obvious by looking to Fig. 1.

As seen with the damping matrix in Eq.(2), the additional two last rows and columns pertaining to soil dampings, makes the total system damping matrix to be in essence nonclassical, i.e., non-proportional to mass and stiffness matrices. Of course, it can be assumed to be proportional just as an approximate presumption.

THE MODAL ANALYSIS

Equation 1 can be rewritten as:

$$[M]\{\ddot{U}\}+[C]\{\dot{U}\}+[K]\,\{U\}=p(t) \tag{5}$$

in which:

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

$$\{p(t)\} = \begin{Bmatrix} \{m\} & m_b + \sum_{1}^{n} m_i & \sum_{1}^{n} m_i h_i \end{Bmatrix}^T \ddot{u}_g(t) \qquad (6)$$

It has been shown in reference (Alibabaei, 2014) that when the damping matrix in Eq. (5) is not proportional to mass and stiffness, response to the base acceleration is as follows:

$$\{\mathrm{U}\} = \sum_{j=1}^{N} \left[\{\alpha_j\} V_j(t) + \{\beta_j\} \dot{D}_j(t) \right] \qquad (7)$$

in which $V_j(t)$ and $\dot{D}_j(t)$ are the pseudo-velocity and the relative velocity of the jth mode equivalent SDF system with natural frequency p_j and damping ratio ξ_j and $\{\alpha_j\}$ and $\{\beta_j\}$ are vectors of modal response distributions.Also, **t**he base shear is computed as the summation of lateral story forces as:

$$V^b(\mathrm{t}) = \sum_{\mathrm{j}=1}^{\mathrm{N}} \left[(m_j^{\alpha}) p_j V_j(\mathrm{t}) + (m_j^{\beta}) p_j \dot{D}_j(\mathrm{t}) \right] \qquad (8)$$

in which m_j^{α} and m_j^{β} are modal mass factors corresponding to the pseudo-velocity and the relative velocity of the jth mode.

DESIGN ASSUMPTIONS

For the purposes of this study, special steel moment frame structures being 1, 2, 4, 6, ..., 18 and 20 story building are designed. The frames have three bays bothbays each spanning 5m.The floor to floor heights are 3m. The residential buildings are designed according to ASCE 7-10 (ASCE 7-10, 2010) and AISC-ASD(AISC-ASD, 2002). The seismicity of the region is considered to be very high with the effective peak acceleration at the ground surface to be 0.35g. Two types of underlying soils are considered: a soft soil (soil type D (ASCE 7-10, 2010)) and a very soft soil (soil type E).Their characteristics are given in Table 1.

Table 1.Characteristics of the soil types.

Soil type	Shear wave velocity $V_s(m/s)$	Unit mass $\rho(kg/m^3)$	Poisson's ratio N	Bearing capacity (kgf/cm^2)
D	250	1800	0.4	2
E	125	1700	0.45	1.5

Eleven buildings with the mentioned number of stories are designed with fixed bases for each soils type. Therefore, totally 22 buildings are considered in this study. For 1 to 4-story buildings single footings and for 6 to 20-story structures strip foundations are designed. The design spectra are according to ASCE 7 for each soil type(ASCE 7-10, 2010).

For time-history analysis of the buildings under study, earthquake records with the following characteristics are selected out of the PEER NGA database (PEER NGA database, 2013) : 6.5≤ Magnitude ≤ 7.5, soil type is whether D or E. Two groups of earthquakes are selected for each building on each soil type regarding the epicentral distance, R. Group one, the near-field earthquakes with R ≤ 20 km, and group two, the far-field earthquakes with 20< R ≤ 50 km. Then the records are scaled according to ASCE7-10 (ASCE 7-10, 2010), such that their response spectra does not fall below the design spectrum of Fig. 2 between 0.2T and 1.5T, where T is the fundamental period of the fixed- base buildings. Then 10 records in each distance group with scale factors closer to unity are retained for dynamic analysis of the same buildings.

MODELING OF SOIL-STRUCTURE INTERACTION

A stick model of the flexible-base 2D frames of the buildingsis developed in Matlab. This model is necessary for nonclassical analysis because such an analysis is not possible in commercial engineering softwares. The flexible base is modeled using a rigid foundation being in dimensions equivalent to the actual foundations of the 2D frames. For strip foundations (in 6 to 20-story buildings), the same dimensions are used. For single footings (in 1 to 4-story buildings), length of the foundation element is taken to be equal to sum of the single foundation lengths in the corresponding 2D frames. Width of the foundation in this case is mean of the foundation widths in the corresponding frame. It can be shown that the mentioned assumptions are appropriate for the responses targeted in this study(Alibabaei, 2014).

The base of the model rests on springs and dampers along the two degrees of freedom in plane as shown in Fig. 2. Characteristics of the springs and dampers are given in the following sections.

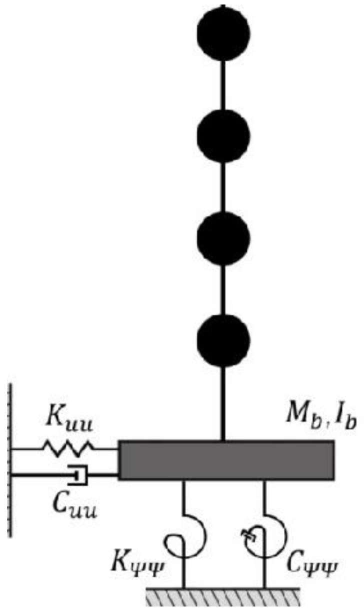

Figure2. Configuration of the stick model

STIFFNESS OF THE SOIL SPRINGS

The stiffness of soil springs is a function of the dynamic shear modulus of soil,G. The dynamic shear modulus can be much smaller than the static shear modulus,G_0, because of the large strains that develop in soil during an earthquake.The ratio G/G_0depend on the soil type and the effective peak acceleration of ground motion at the ground surface. Values of $G/G_0$for the two soil types are used according to ASCE 7-10 (ASCE 7-10, 2010).

According to ASCE41-13 (ASCE/SEI 41-13, 2013), before determining the spring stiffnesses, condition of the foundation, being flexible or rigid with regard to the underlying soil, must be determined. The above criterion results in all of the foundations of this study resting on the soil type E to be rigid. For the soil type D, only foundations of 1, 2 and 4-story buildings prove to be rigid. For flexible foundations, a uniformly distributed vertical spring is utilized ASCE41-13 (ASCE/SEI 41-13, 2013). For rigid foundations, coupling of vertical and rocking degrees of freedom is taken into account using a nonuniform distribution of vertical springs.

For this purpose, each foundation is divided to interior and exterior zones. The exterior zones are two rectangles, one at each end of the foundation, with a length of$B/6$(B=foundation width) and a width equal to that of the foundation (ASCE/SEI 41-13, 2013).

DAMPING

Design spectra are given for a damping ratio of 0.05, as common. For SSI applications, the spectral values will be needed for other damping ratios. This is usually done using a spectral reduction factor *RF* (Alibabaei, 2014).The damping coefficients in the horizontal and rocking degrees of freedom have been given as(Gazetas, 1991):

$$C_{uu} = pV_{la}A_b$$
$$C_{..} = pV_{la}I_{by}\tilde{c}_{ry} \qquad (12)$$

where p is the unit mass of soil, A_b is the area of foundation in plan, I_{by} is the moment of inertia of the foundation in plan about the transverse axis,$V_{la}$is a wave velocity equal to $\frac{3.4\,V_s}{\pi(1-\nu)}$ with V_s being the shear wave velocity, and $\tilde{c}_{ry}$ is a coefficient beginning from zero for the static case and tending to unity for very large excitation frequencies.

THE ANALYSIS RESULTS

RESPONSES IN THE FUNDAMENTAL MODE

A modal time history analysis is accomplished in the section. All of the response parameters are shown versus the fixed-base period of each building. Figure 3 shows the averaged maximum story drift ratios.

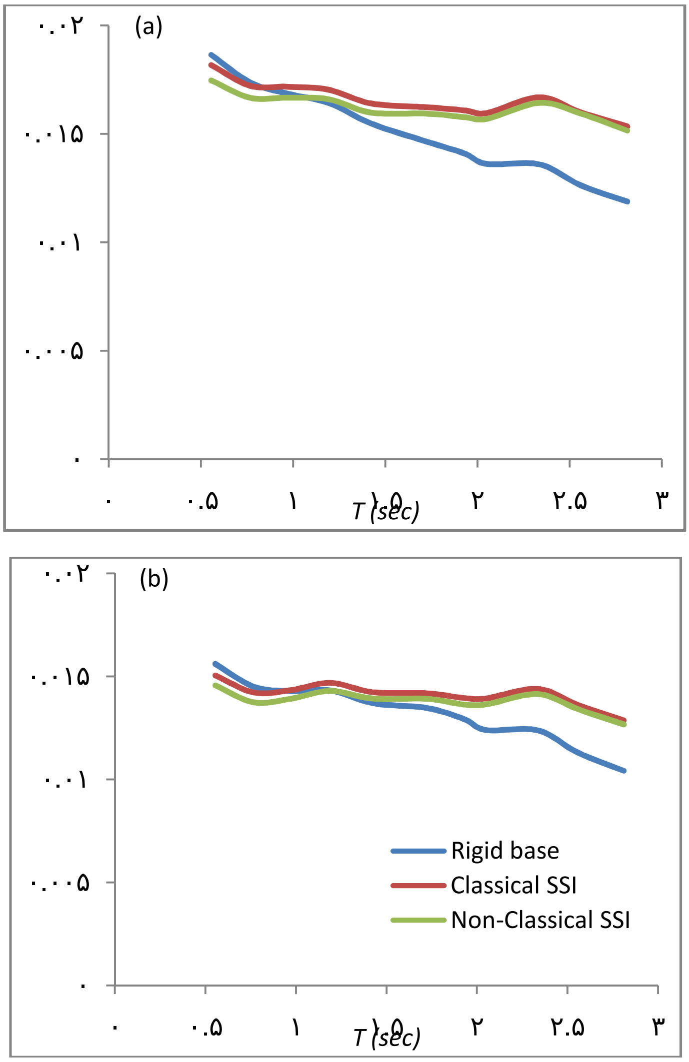

Figure3. The averaged maximum story drift ratios of each building. (a) Soil type D,

near field earthquakes;(b) Soil type D, far field earthquakes

According to Fig. 3, from a period of about 1 second, story drifts of the flexible-base models overtake those of the fixed-base model considerably, for both categories of earthquakes. The relative difference of drifts between models increases with height such that for the soil types D and E it reaches to about 33% and 56%, respectively, for larger periods. For periods smaller than 1 s, difference between the drifts calculated by the three models is small and effect of SSI on displacements is negligible. It is interesting that in the same period range, the nonclassical analysis results in drifts smaller than those of the fixed-base model.The maximum base shear of each system normalized to its weight is shown in Figure4.

Figure 4 shows that SSI decreases the base shear on both soil types. The reduction is important from the same 1 s period mentioned in drift analysis. The more rigorous nonclassical analysis procedure is similar in results to the fixed-base case for periods smaller than 1 s and to the classical procedure for larger periods. The base shear reduction is up to 23% and 38% for taller buildings. For periods smaller than 1 s, soil-structure interaction should not be taken into account for seismic analysis of structures similar to the ones of this study.

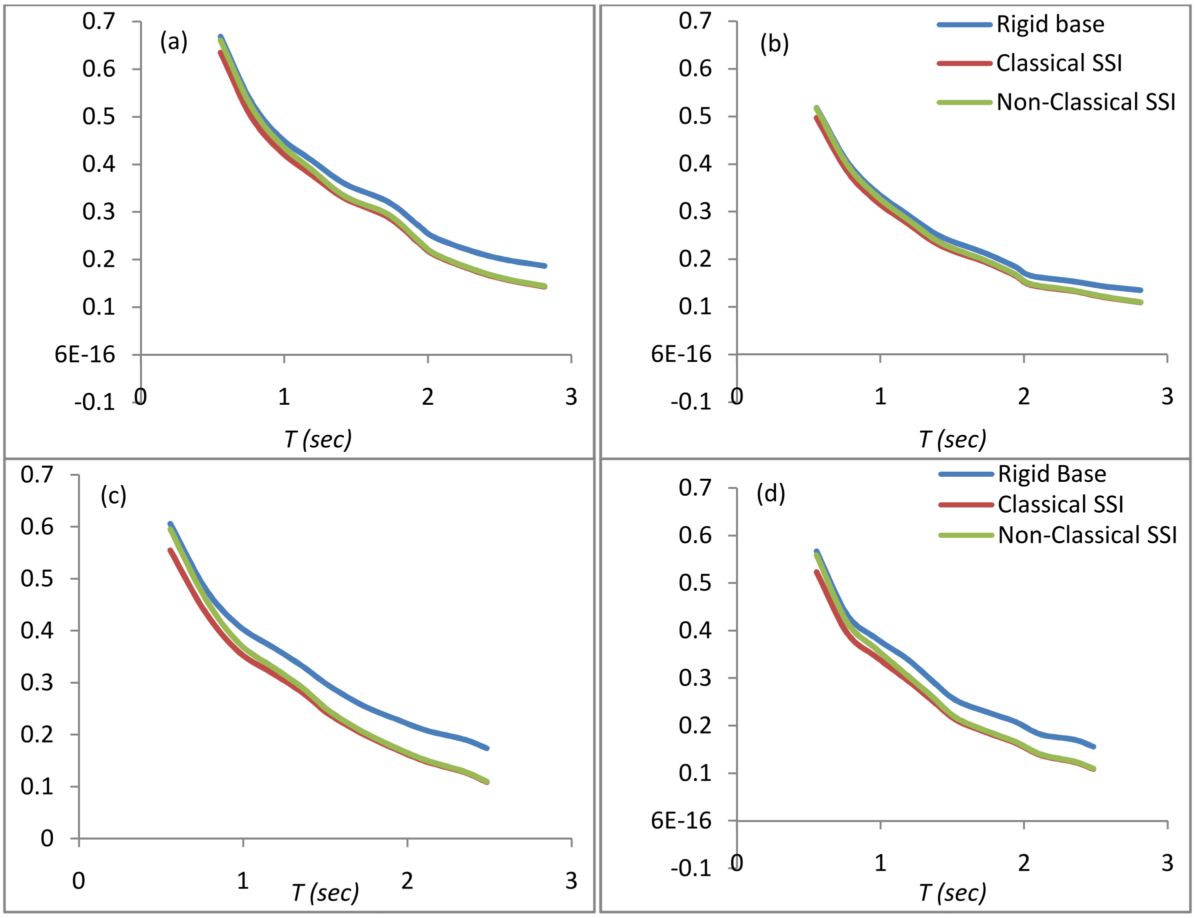

Figure4. Averaged maximum base shear normalized to the building weight. (a)Soil type D, near field earthquakes.(b)Soil type D, far field earthquakes. (c) Soil type E, near field earthquakes. (d) Soil type E, far field earthquakes

RESPONSES IN THE HIGHER MODES

In the current section sum of the response corresponding to all other modes (called the higher modes) is illustrated. The averaged maximum values of base shear normalized to the building weight are shown in Figure5 for the three analysis cases. The values have been also averaged between near and far field earthquakes.

Based on Fig. 5, it can be said that response in the higher modes can equally be calculated using classical or nonclassical analysis. Therefore, use of nonclassical analysis is not necessary for the higher modes. On the other hand, accounting for SSI in the higher modes is important for systems with fixed-base fundamental periods larger than 2.5 s when calculating displacements (not shown for brevity) and larger than 2 s when deriving the base shear. In such ranges, higher mode displacements increase and base shears decrease considerably due to SSI.

CONCLUSIONS

In this study, several structures having one to twenty stories resting on two types of soils, being medium and soft, were analyzed each one under two suits of ten consistent and scaled earthquake motions specific to that structure recorded at near and far distances. Modal time history analysis was accomplished in its classical and non-classical versions. It was shown that the relative difference of displacements between the associated fixed-base and flexible-base models became important from a period of about 1 s and increased with height, and was larger for the softer soil. SSI decreased the base shear on both soil types. The reduction was important again from a period of 1s.For periods smaller than 1s, soil-structure interaction should not be taken into account for seismic analysis of moment frame structures resting on surface foundations. Also, use of nonclassical SSI analysis is not necessary for the higher modes. Accounting for SSI in the higher modes is important for systems with fixed-base fundamental periods larger than 2 s.

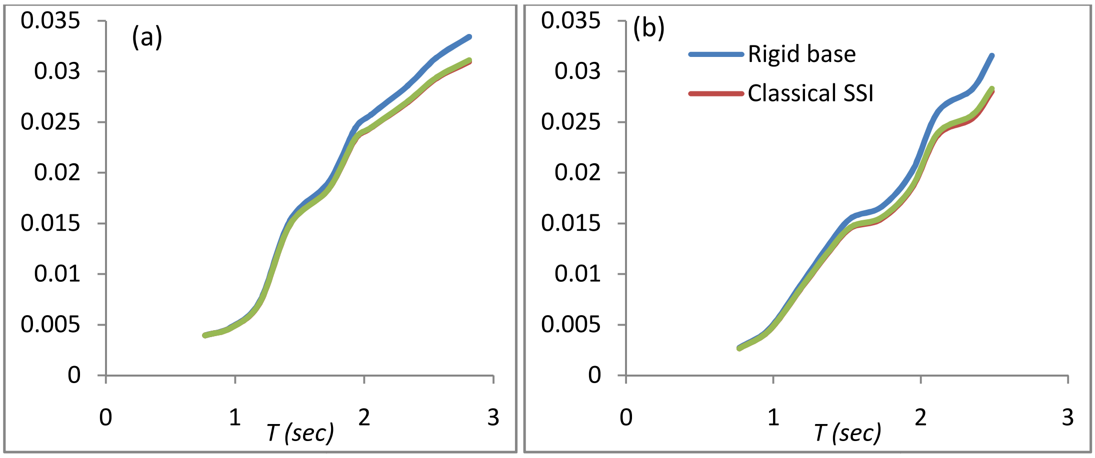

Figure5. The averaged maximum base shear corresponding to the higher modes, normalized to the building weight; (a) Soil type D; (b) Soil type E

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