Estimation of Inelastic Displacement Demands of Flexible-Base Structures on Soft Soils

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Abstract

This study aims to develop efficient tools for performance-based seismic design of soil-structure interaction (SSI) systems on soft soils. To simulate the SSI effects, linear and non-linear "equivalent fixed-base single-degree-of-freedom" (EFSDOF) oscillators as well as a sway-rocking SSI model were adopted. The nonlinear dynamic response of around 10,000 SSI models and EFSDOF oscillators having a wide range of fundamental periods, target ductility demands, and damping ratios were obtained under a total of 20 seismic records on soft soil sites. Based on the results of this study, a practical method is developed for estimating the base shear and maximum displacement demands of a non-linear single-degree-of-freedom structure on soft soil deposits. In the proposed procedure, the effect of frequency content of ground motions is considered by normalizing the period of structures by the spectral predominant periods of the SSI systems, while the nonlinear EFSDOF models are used to improve the computational efficiency.

Keywords: Soil-structure interaction; soft soil; displacement demands; response-history analysis; frequency content

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1 Introduction

The preliminary design of typical building structures in current seismic design codes and provisions is mainly based on elastic spectrum analysis, where the base shear and displacement demands of nonlinear systems are estimated by using modification factors such as the ductility reduction factor R_μ and inelastic displacement ratio C_μ. However, structures built on soft soil deposits exhibit noticeably different seismic responses compared to those located on firm sites when subjected to earthquake excitations. Firstly, the frequency content of seismic records for soft soil conditions may vary significantly from one site to another. Secondly, the “fixed-base” assumption that buildings are rigidly supported at their base is not appropriate due to the lower soil stiffness.

A number of studies have been carried out to investigate the effect of frequency content of earthquake ground motions on the structural response of structures. Rathje et al. (1998) evaluated several scalar-valued parameters that characterised the frequency content of an input motion using 306 acceleration records from 20 earthquakes in active plate-margin regions. They found that a mean period, averaged from a range of periods from 0.05 to 4sec in the Fourier spectrum of an acceleration record, was the most reliable parameter when used to normalise the period of vibration. Xu and Xie (2004) and Ziotopoulou and Gazetas (2010) proposed that the periods of an acceleration response spectrum should be normalised with respect to the spectral predominant period (corresponding to peak ordinate) in order to capture the peak spectral response. Similar suggestions have been made to improve the velocity (e.g. Mavroeidis et al., 2004; Xu and Xie, 2007) and displacement (Maniatakis and Spyrakos, 2012) response spectra, and also to modify the ductility reduction factor (e.g. Miranda and Bertero, 1994; Miranda and Ruiz-Garcia, 2002; Gillie et al., 2010), and inelastic displacement ratio (e.g. Miranda, 2000; Ruiz-García and Miranda, 2006; Iervolino et al., 2012) for nonlinear systems. However, all of these studies were restricted to fixed-base building systems and, therefore, the effects of soil-structure interaction (SSI) were not considered.

The implementation of SSI into seismic design has received a considerable amount of attention in recent years. Takewaki (1998) proposed a semi-explicit ductility-based design method for flexible-base multi-storey building based on equivalent linearization. Ghannad and Jahankhah (2007) studied the inelastic seismic demands of flexible-base structures and concluded that using the ductility reduction factor derived on the basis of the "fixed-base" assumption for seismic design of SSI systems could lead to non-conservative design solutions. More recently, Lu et al. (2016) proposed a performance-base design procedure for flexible-base multi-storey buildings, based on response-history analysis using synthetic spectrum-compatible earthquakes in accordance with code-specified soil site classifications. In their proposed procedure, they explicitly included the characteristic period, which is defined as the transition period from the acceleration-controlled to the velocity-controlled segment of a 5% damped design response spectrum of a design ground motion. The combined effects of SSI and frequency content of near-fault ground motion were extensively studied by Khoshnoudian and Ahmadi (Khoshnoudian and Ahmadi, 2013; Ahmadi and Khoshnoudian, 2015; Khoshnoudian and Ahmadi, 2015). Kojima and Takewaki (2016) derived a closed-form solution of the response of a flexible-base elastic-plastic structure subject to fling-step near-fault ground motion represented using a double impulse, based on the work by Kojima and Takewaki (2015). However, the explicit inclusion of the effect of the frequency content of ordinary ground motions recorded on soft soils in the seismic design of SSI systems is still an area of uncertainty.

This paper addresses several issues concerning seismic design of structures on soft soil deposits by studying elastic and constant-ductility response spectra of SSI systems. A new method is proposed to estimate the displacement demands of flexible-base buildings on the basis of response spectra for fixed-base single-degree-of-freedom oscillators, which also enables the effect of frequency content of records on soft soil to be taken into account.

2 Models and parameters

2.1 Soil-structure interaction model

To investigate the seismic performance of structures on soft soil profiles, a simplified SSI model was adopted in the present study, as depicted in Figure 1. The superstructure was modelled as a single-degree-of-freedom (SDOF) oscillator having a mass of m_s, a mass moment of inertia of J_s, and a height of h_s. An elastic-perfectly plastic lateral force-displacement response, with an initial stiffness of k_s and a lateral strength of V_y, was assumed for the oscillator. The adopted model can simulate the seismic behavior of non-deteriorating structural systems such as moment resisting steel frames. The level of inelasticity within the structure was controlled by a ductility ratio of $\mu=u_m/u_y$, with u_m being the displacement demand and u_y the yielding displacement. This ductility ratio can be associated with either a ductility reduction factor $R_\mu=V_e/V_y$ or an inelastic displacement ratio $C_\mu=u_m/u_e$, where V_e and u_e are the elastic base shear and maximum elastic displacement demand, respectively.

The dynamic interaction between foundation and soil was simulated using the cone model on the basis of idealizing a homogeneous soil half-space under a rigid circular disk (having a mass of m_f, mass moment of inertia J_f, and radius of r) as a semi-infinite truncated cone (Ehlers, 1942). The soil medium is characterised by a mass density of ρ, Poisson's ratio of ν, shear wave velocity of v_s, and dilatational wave velocity of v_p. This simplified SSI model enables the frequency-dependent global behaviour of the soil-foundation system (i.e. foundation swaying and rocking motions) to be solved in the time domain. The adequacy of this SSI model to predict the seismic performance of non-linear systems on soft soil was investigated by Lu et al. (2016).

The dynamic properties of the superstructure relative to those of the overlying soil medium can be described using the following dimensionless parameters:

- $a_0=2\pi h_s/(T_s v_s)$ is the structure-to-soil stiffness ratio, with T_s being the fundamental period of the superstructure in its fixed-base condition. It was shown that a_0 generally varies from zero for fixed-base buildings, to a value of three for buildings located on very soft soil deposits (Lu et al., 2016).
- $s=h_s/r$ is the slenderness ratio of the superstructure. For conventional building structures the slenderness ratio is usually in the range of 1 to 4.
- $\overline{m}=m_s/(\rho h_s r^2)$ is the structure-to-soil mass ratio. Note that for a multi-storey building, m_s is the effective seismic mass and h_s is the effective height of the building. Alternatively, one can also use the total mass and total height of the building to calculate $\overline{m}$.

In the current study, the value of $\overline{m}$ was set to 0.5, the foundation mass m_f was assumed to be ten percent of the effective mass of the superstructure m_s, and the soil Poisson's ratio was taken as 0.5 for very soft soil in undrained conditions. The elastic structural energy dissipation was measured using a viscous damping ratio of ξ_s=0.05 and the soil hysteretic damping ratio ξ_g was also set to 0.05.

2.2 Equivalent fixed-base SDOF oscillator

It is common practice in preliminary design of conventional building structures to replace a soil-structure interaction system by an equivalent fixed-base single-degree-of-freedom (EFSDOF) oscillator for facilitating SSI analyses. For linear-elastic SSI systems, the effective period T_{ssi} and damping ratio ξ_{ssi} of the EFSDOF representative of an SSI system can be calculated according to Maravas et al. (2014) as follows:

$$T_{ssi} = 2\pi / \sqrt{\frac{\chi}{1+4\xi_{ssi}^2}}\ ,\ \xi_{ssi} = \chi\left[\frac{\xi_h}{\omega_h^2\left(1+4\xi_h^2\right)} + \frac{\xi_\theta}{\omega_\theta^2\left(1+4\xi_\theta^2\right)} + \frac{\xi_s}{\omega_s^2\left(1+4\xi_s^2\right)}\right] \tag{1}$$

where χ is defined as:

$$\chi = \left[\frac{1}{\omega_h^2\left(1+4\xi_h^2\right)} + \frac{1}{\omega_\theta^2\left(1+4\xi_\theta^2\right)} + \frac{1}{\omega_s^2\left(1+4\xi_s^2\right)}\right]^{-1} \tag{2}$$

while the frequencies ω_h, ω_θ and damping ratios ξ_h, ξ_θ (including both radiation damping and soil material damping) are calculated according to:

$$\omega_h = \sqrt{\frac{\alpha_h k_h - m_f \omega^2}{m_s}}, \omega_\theta = \sqrt{\frac{\alpha_\theta k_\theta - (J_f + J_s)\omega^2}{m_s h_s^2}}, \xi_h = \frac{\omega r}{v_s}\frac{\beta_h}{2\alpha_h}, \xi_\theta = \frac{\omega r}{v_s}\frac{\beta_\theta}{2\alpha_\theta} \tag{3}$$

where ω is the frequency of vibration, and k_h and k_θ are, respectively, the swaying and rocking static foundation stiffness. The coefficients α_h, α_θ, β_h, β_θ are frequency-dependent parameters that can be calculated based on the closed-form expressions proposed by Veletsos and Verbič (1973).

To take into account the nonlinear behaviour of the structural system, either a nonlinear EFSDOF or an equivalent linear EFSDOF oscillator can be used to simplify the SSI procedures. In additional to T_{ssi} and ξ_{ssi}, a nonlinear EFSDOF oscillator is characterised by an effective ductility ratio of μ_{ssi} defined as (Avilés and Pérez-Rocha, 2003):

$$\mu_{ssi} = \lambda^{-2}\left(\mu_s - 1\right) + 1 \tag{4}$$

where $\lambda = T_{ssi}/T_s$ is the period lengthening ratio evaluated for linear systems. Figure 2 shows the elastic and inelastic EFSDOF oscillators corresponding to a SSI system.

Replacing a nonlinear inelastic SSI system with a linear EFSDOF oscillator is usually done by means of equivalent linearization. Esmaeilzadeh Seylabi et al. (2012) developed such a linear model (having a period of T_{eq} and viscous damping ratio of ξ_{eq}) by ensuring that its displacement demands approximated those of the corresponding SSI system (model shown in Figure 1) based on results of a response history analysis. The following expressions were proposed to estimate the equivalent linear period and damping ratio:

$$T_{eq}/T_s \text{ or } \xi_{eq} = \left(c_1 + c_2\mu_s + c_3\mu_s^2\right)\exp\left(-c_4 s\right) + \left(c_5 + c_6 s + c_7 s^2\right)\exp\left(-c_8\mu_s\right) \tag{5}$$

where c_1 to c_8 are constants from regression analysis available in Esmaeilzadeh Seylabi et al. (2012). Note that ξ_{eq} in Eq. (5) is expressed in percentage whereas damping ratios described elsewhere in this study take their actual values unless stated otherwise. Moghaddasi et al. (2015) suggested a methodology to derive linear period and damping ratio, given by Eq. (6), by transforming the nonlinear EFSDOF oscillator into an equivalent linear model utilizing existing methods for fixed-base systems. It should be noted that for linear systems, Eq. (6) reduces to Eq. (1).

$$T_{eq}/T_s = \lambda\sqrt{\mu_{ssi}}\,,\;\; \xi_{eq} = \xi_{ssi} + \sqrt{\frac{\mu_{ssi}}{\mu_s}\frac{\left(\mu_s - 1\right)}{\left(\mu_{ssi} - 1\right)}}^{\,-3} \mu_s^{-0.81}\xi_s + \frac{1 - 1/\sqrt{\mu_{ssi}}}{\pi} \tag{6}$$

2.3 Proposed methodology

In order to compare the effectiveness of the nonlinear and equivalent linear EFSDOF oscillators in predicting the seismic demands of SSI systems, the simplified SSI model illustrated in Figure 1 was used as the benchmark model with its equation of motion given by Eq. (7).

$$[M]\{\ddot{u}(t)\} + [C]\{\dot{u}(t)\} + [K]\{u(t)\} = -[M]\{R\}\ddot{u}_g(t) \tag{7}$$

where u_g is the ground displacement record. Over-dot indicates the derivative with respect to time. The mass M, damping C, and stiffness K matrices as well as the displacement u and influence coefficient R vectors are respectively given by:

$$[M]=\begin{bmatrix} m_s & 0 & 0 & 0 \\ & m_f+c_h\xi_g/\omega_{ssi} & 0 & 0 \\ & & J_s+J_f+M_\theta+c_\theta\xi_g/\omega_{ssi} & -c_\theta\xi_g/\omega_{ssi} \\ \text{Sym.} & & -c_\theta\xi_g/\omega_{ssi} & M_\varphi+c_\theta\xi_g/\omega_{ssi} \end{bmatrix} \tag{8}$$

$$[C]=\begin{bmatrix} c_s & -c_s & -c_s h_s & 0 \\ & c_h+c_s+2k_h\xi_g/\omega_{ssi} & c_s h_s & 0 \\ & & c_\theta+c_s h_s^2+2k_\theta\xi_g/\omega_{ssi} & -c_\theta \\ \text{Sym.} & & & c_\theta \end{bmatrix} \tag{9}$$

$$[K]=\begin{bmatrix} k_s & -k_s & -k_s h_s & 0 \\ & k_h+k_s & k_s h_s & 0 \\ & & k_s h_s^2+k_\theta & 0 \\ \text{Sym.} & & & 0 \end{bmatrix} \tag{10}$$

$$\{u\}=[u_{ssi},u_h,\theta,\varphi]^T, \quad \{R\}=[1,1,0,0]^T \tag{11}$$

where k_h, k_θ and c_h, c_θ are static foundation stiffness and high-frequency damping coefficient for the sway and rocking motions, respectively. The mass moment of inertia M_θ and M_φ (in the rotational degree-of-freedom φ) are used to account for soil incompressibility and frequency-dependency in the rocking degree-of-freedom θ ; u_{ssi} is the displacement of the structural mass relative to the ground, while u_h is the foundation swaying displacement relative to the ground. The equations of motion for the SSI model and EFSDOF oscillators were solved in the time domain using the methods presented in Lu et al. (2016). A suite of 20 ground motions recorded on soft soil deposits were employed in the study, as listed in Table 1.

3 Elastic response spectra

Most of the current code design acceleration response spectra, obtained by averaging a number of actual response spectra, have a constant acceleration plateau that encompasses the peak design seismic forces within a representative SDOF building. This flat segment is generally larger and defined by a higher corner period for softer soil conditions because soft soil tends to amplify the long-period components of a ground motion. However, many studies have shown sharp peaks in the response spectra of earthquake records on soft soil deposits rather than a flat shape (e.g. Xu and Xie, 2004; Ziotopoulou and Gazetas, 2010; Maniatakis and Spyrakos, 2012). This inconsistency is the result of averaging dissimilar individual response spectrum (Ziotopoulou and Gazetas, 2010). An example is illustrated in Figure 3, where the response spectra for ground motions recorded at three soft soil sites during the 1989 Loma Prieta earthquake exhibit noticeably different peaks at well-separated periods.

The issue concerning unrealistic averaging may be resolved by using the bi-normalised response spectra where the period of vibration of a system is normalised with respect to a predominant period of T_P corresponding to the peak of a response spectrum. To demonstrate the efficiency of the proposed solution, a total of 1009 cases of seismically-excited structures on soft soils were studied. The details of the these seismic records are provided in Ziotopoulou and Gazetas (2010). Figure 4 compares the conventional and bi-normalised response spectra of the selected records. It is shown that the averaged bi-normalised spectrum can preserve the peak acceleration, which is roughly 1.5 times that corresponding to the constant acceleration plateau of the conventional spectrum. Using the same idea for SSI systems, it is suggested that the effective linear period of vibration of a system T_{ssi} should also be normalised by T_P.

Figure 5 compares the averaged bi-normalised acceleration and displacement spectra obtained using the SSI models and EFSDOF oscillators for the twenty ground motions listed in Table 1. Despite some under-prediction of the accelerations and displacements of the SSI systems having an effective viscous damping ratio of ξ_{ssi} greater than ten percent, it is shown that the EFSDOF oscillators are an excellent substitute for flexible-base buildings. The bi-normalised response spectra may be described by the following expressions which are also plotted in Figure 5:

$$\text{SA/PGA} = \begin{cases} \exp\left(a_1 T/T_P\right) & T/T_P \le 1 \\ a_2\left(T/T_P\right)^{a_3} & T/T_P > 1 \end{cases}, a_1 = -0.51\ln\zeta + 2.14, a_2 = \exp\left(a_1\right), a_3 = 0.18\ln\zeta - 1.53 \quad (12)$$

$$\text{SD/PGD} = \begin{cases} b_1\left(T/T_P\right)^{b_2} & T/T_P \le 1 \\ \exp\left(b_3 T/T_P\right) & T/T_P > 1 \end{cases}, b_1 = 6.59\zeta^{-0.47}, b_2 = -0.42\ln\zeta + 2.06, b_3 = \ln\left(b_1\right) \quad (13)$$

where SA and SD are, respectively, spectral acceleration and spectral displacement, and PGD is peak ground displacement. $\zeta=100\xi$ is the viscous damping in percentage and has a practical range from five to twenty (ASCE, 2010). For fixed-base systems the period (T) and damping ratio (ξ) in Eqs (7) and (8) are substituted by T_s and ξ_s , whereas for SSI systems they are replaced by T_{ssi} and ξ_{ssi}.

4 Inelastic displacement demands

Since an inelastic SSI system can be replaced by either a nonlinear EFSDOF or an equivalent linear EFSDOF oscillator, it is necessary to compare the effectiveness of these models. In this paper NEFSDOF, LESDOF1 and LESDOF2 respectively denote the nonlinear model, and the two equivalent linear models proposed by Esmaeilzadeh Seylabi et al. (2012) and Moghaddasi et al. (2015). When using LESDOF1, only the displacement demands are representative of those of the corresponding SSI system; the acceleration and velocity demands do not represent the actual behaviour of the SSI system. Therefore, in this section only the displacement demands of flexible-base inelastic buildings are compared with those of their EFSDOF oscillators. These displacement demands are measured relative to the ground and encompass both structural deformations and foundation rigid-body movements (i.e. swaying and rocking motions). The properties of the benchmark SSI models (defined mainly by a_0 and s) and their corresponding EFSDOF oscillators (defined by the period lengthening ratio of T/T_s, effective damping ratio of ξ, and effective ductility ratio of μ) are summarised in Table 2. Note that NEFSDOF systems are governed by T_{ssi}, ξ_{ssi} and μ_{ssi} whereas LEFSDOF1 and LESDOF2 systems are mainly a function of T_{eq} and ξ_{eq}.

The displacement demands of the three EFSDOF oscillators subjected to the 360° component of the horizontal motion recorded at Larkspur Ferry Terminal during the 1989 Loma Prieta earthquake are compared with those of the SSI models in Figure 6. In general, it is shown that nonlinear EFSDOF oscillators perform much better than linear EFSDOF systems in predicting the displacement demands of SSI systems. This is especially evident in SSI systems with low effective elastic viscous damping ratio (i.e. ξ_{ssi}<10%) and high structural ductility demand (i.e. μ_s=6). For SSI systems having a ξ_{ssi} value of around 20%, using the NEFSDOF underestimates the displacement demands of the actual SSI systems in the intermediate-to-long-period range. The results also indicate that for the linear EFSDOF oscillators, LESDOF1 provides a better estimation of displacement demands compared to LESDOF2 for SSI systems having a ξ_{ssi}=20%, whereas for lightly damped systems the trend is reversed. Since it was shown that in general using the nonlinear EFSDOF oscillators (NEFSDOF) leads to better estimation of the seismic demands of SSI systems compared to the linear EFSDOF alternatives, the following section will be focused on the application of the nonlinear EFSDOF oscillators in the performance-based seismic design of SSI systems.

5 Ductility reduction factor and inelastic displacement ratio

In the preliminary design of building structures, it is generally desirable to calculate strength or displacement demands by applying modification factors to the elastic response spectra instead of carrying out cumbersome and computationally expensive non-linear response history analyses. In this section, the ductility reduction factor R_μ and the inelastic displacement ratio C_μ of SSI systems are studied using NEFSDOF oscillators. The accuracy of the estimated values is also investigated by using the more accurate SSI model shown in Figure 1. While the displacement demands obtained from NEFSDOF models encompass the rigid-body foundation movements, the obtained ductility ratios μ_{ssi} can be directly used for calculating R_μ and C_μ. In this study, the predominant period T_g, corresponding to the peak ordinate of a velocity response spectrum for a damping ratio of ξ_{ssi}, was adopted to normalise the effective period of SSI systems T_{ssi}, as suggested by Miranda and Bertero (1994) and Miranda and Ruiz-Garcia (2002).

Figure 7 presents the mean values of R_μ and C_μ for the 20 ground motions listed in Table 1. In this figure, the results of SSI models with different combinations of a_0, s and ξ_g (in order to achieve an identical damping ratio value of ξ_{ssi}=10% or 20%) are compared with those of the NEFSDOF oscillators having the same elastic damping ratio. It is shown that for SSI systems with low effective elastic damping ratios (i.e. ξ_{ssi}<10%), the NEFSDOF oscillators provide a good estimation of both R_μ and C_μ of the benchmark SSI models. However, NEFSDOF systems overestimate and underestimate, respectively, R_μ and C_μ of the SSI models with a relatively higher effective elastic damping ratio of ξ_{ssi}=20%, which is the upper-bound limit specified in most seismic provisions (e.g. ASCE, 2010). The underestimation of C_μ is a direct result of the under-prediction of the inelastic displacement demands explained with reference to Figure 6 in the previous section.

In order to improve the performance of the NEFSDOF oscillators for predicting the ductility reduction factor and the inelastic displacement ratio of SSI systems, modifications to the NEFSDOF systems are required. Note that C_μ can be calculated by dividing μ by R_μ for an elastic-perfectly plastic force-displacement relation. Therefore, for a given μ value, C_μ is inversely proportional to R_μ. Based on the results of this study, a correction factor can be defined as follows:

$$\alpha_\xi = \frac{R_{\mu,\mathrm{NEFSDOF}}}{R_{\mu,ssi}} = \frac{C_{\mu,ssi}}{C_{\mu,\mathrm{NEFSDOF}}} \qquad (14)$$

The correction factor α_ξ was calculated for individual SSI systems having ten different effective elastic damping ratios increasing from 11% to 20% at an interval of 1%. An example for the results corresponding to μ_{ssi}=5 is presented in Figure 8(a) which shows higher values of α_ξ at higher effective damping levels. The variation of α_ξ with the normalised period T_{ssi}/T_g can be described using a piecewise approximation given by Eq. (15). The accuracy of the proposed equation to calculate the correction factor α_ξ is demonstrated in Figure 8(b).

$$\alpha_\xi = \begin{cases} \dfrac{\mathrm{c}-1}{0.4}\dfrac{T}{T_g}+1 & \dfrac{T}{T_g} \le 0.4 \\ \mathrm{c} & 0.4 < \dfrac{T}{T_g} \le 0.9 \\ \left(1.5-\dfrac{T}{T_g}\right)\dfrac{\mathrm{c}-1}{0.6}+1 & 0.9 < \dfrac{T}{T_g} \le 1.5 \\ 1 & \dfrac{T}{T_g} > 1.5 \end{cases} \qquad \mathrm{c} = \mu^{(0.12\ln\xi+0.3)} \qquad (15)$$

By applying the site-dependent correction factor in Eq. (15) to the results of NEFSDOF oscillators, a better prediction of R_μ and C_μ of the SSI systems is obtained, as shown in Figures 8(c)-(f). Using the bi-normalised elastic and inelastic response spectra, derived by the equivalent fixed-base SDOF oscillators, a more realistic estimate of the inelastic seismic demands of flexible-base structures can be made.

The results of this study in general highlight the importance of taking into account the frequency content of the design earthquakes (spectral predominant periods) for seismic design of structures on soft soil conditions. Compared to existing SSI procedures based on equivalent fixed-base SDOF oscillators, the proposed methodology can provide improved estimation of strength and displacement demands of SSI systems (especially for systems with high initial effective damping ratios) by explicitly including the effect of frequency content of ground motions on the seismic response of structures. The outcomes of this study should prove useful in performance-based seismic design and assessment of flexible-base structures on soft soils.

6 Conclusions

This study aimed to develop a more efficient methodology to estimate the base shear and maximum displacement demands of flexible-base structures on soft soils. Based on the results of around 10,000 soil-structure interaction (SSI) systems and EFSDOF oscillators having a wide range of fundamental periods, target ductility demands and damping ratios subjected to a total of 20 ground motions recorded on soft soil sites, the following conclusions were drawn:

- Bi-normalised elastic acceleration and displacement response spectra (with their abscissa normalised with respect to T_P that corresponds to spectral peaks) reflect more realistic seismic demands of linear SSI systems in comparison with the conventional code design spectra.
- The nonlinear EFSDOF oscillator, in general, performed better than the equivalent-linear EFSDOF systems in estimating displacement demands of nonlinear flexible-base structures.
- Normalizing the periods in R_μ and C_μ spectra by the predominant period of T_g, corresponding to the spectral peaks of the elastic velocity spectra, leads to more realistic values of R_μ and C_μ for SSI systems on soft soils.

- Using nonlinear EFSDOF oscillators can provide reliable results for flexible-base structures on the basis of R_μ and C_μ for SSI systems having an effective elastic damping ratio of $\xi_{ssi} \leq 10\%$. However, nonlinear EFSDOF oscillators in general overestimate and underestimate, respectively, R_μ and C_μ for SSI systems having an effective elastic damping ratio of $\xi_{ssi} > 10\%$. To address this issue, a modified nonlinear EFSDOF oscillator was proposed based on site dependent correction factors to improve the prediction of R_μ and C_μ for "highly damped" SSI systems.
- The base shear and displacement demands of a nonlinear flexible-base structure can be estimated accurately by means of the proposed elastic and inelastic spectra derived from response-history analysis on the linear and modified nonlinear EFSDOF oscillators. The proposed methodology can be efficiently used in the performance-based seismic design of flexible-base structures on soft soils by taking into account the SSI effects.

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8 Nomenclature

a_0=structure-to-soil stiffness ratio

C_μ=inelastic displacement ratio

h_s=effective height of a superstructure

J_j= mass moment of inertia

k_j=stiffness

m_j=mass

M_θ=mass moment of internal accounting for soil incompressibility

M_φ=mass moment of internal accounting for frequency-dependency

$\bar{m}$=structure-to-soil mass ratio

r=radius of an equivalent circular foundation

R_μ=ductility reduction factor

s=slenderness ratio of the structure

T_{eq}=period of vibration of an equivalent-linear system

T_g=predominant period corresponding to the peak of a velocity response spectrum

T_j=fundamental period of a system

T_P=predominant period corresponding to the peak of an acceleration response spectrum

u_e=displacement demand of a linear system

u_j=displacement

u_m=displacement demand

u_y=yielding displacement

v_p=dilatational wave velocity within soil medium

v_s=shear wave velocity within soil medium

V_e=base shear demand of a linear superstructure

V_y=base shear strength of a superstructure

α_j=dimensionless spring coefficient for soil impedance

α_ξ=damping correction factor

β_j=dimensionless damping coefficient for soil impedance

λ=period lengthening ratio

μ_j=ductility ratio

ν=soil Poisson's ratio

ξ_j=damping ratio

ξ_{eq}=damping ratio of an equivalent-linear system

ρ=soil mass density

ω_j=circular frequency of vibration

Note that the subscript j is used in a generalised sense to denote s, f, g, ssi, h, and θ that represent, respectively, superstructure, foundation, soil, SSI system, foundation swaying motion and foundation rocking motion.

Figure 1 Soil-structure interaction model

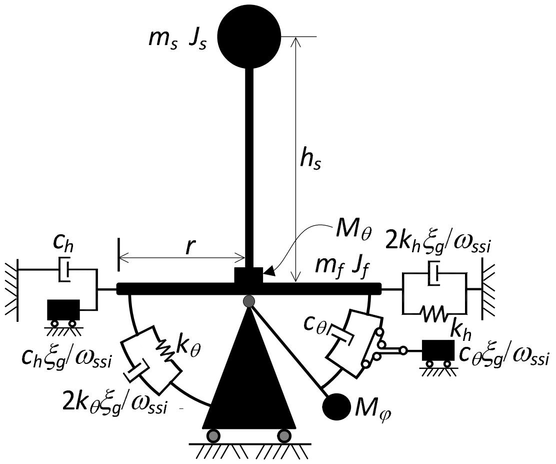

Figure 2 Elastic and inelastic EFSDOF oscillators to design flexible-base structures

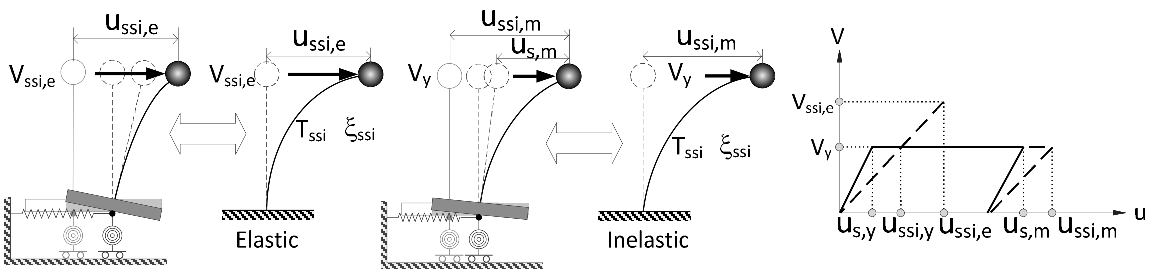

Figure 3 Comparison of 5% damped acceleration spectra for earthquake records from three different stations during the 1989 Loma Prieta earthquake

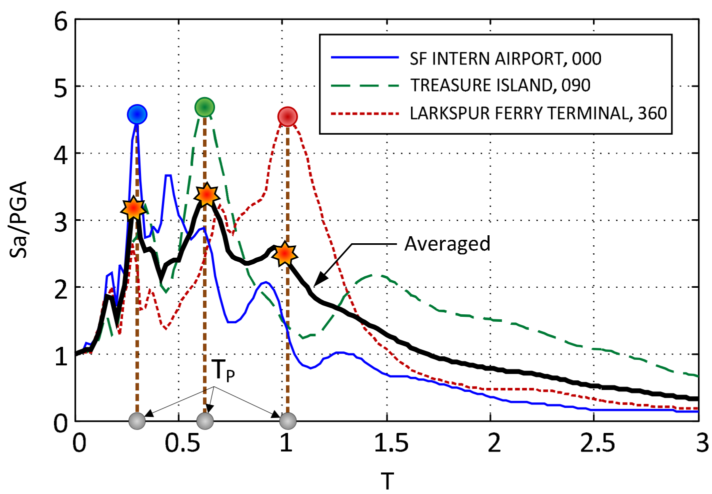

Figure 4 Comparison of conventional and bi-normalised response spectra for a group of earthquakes recorded on soft soils (shaded area envelops all individual spectra)

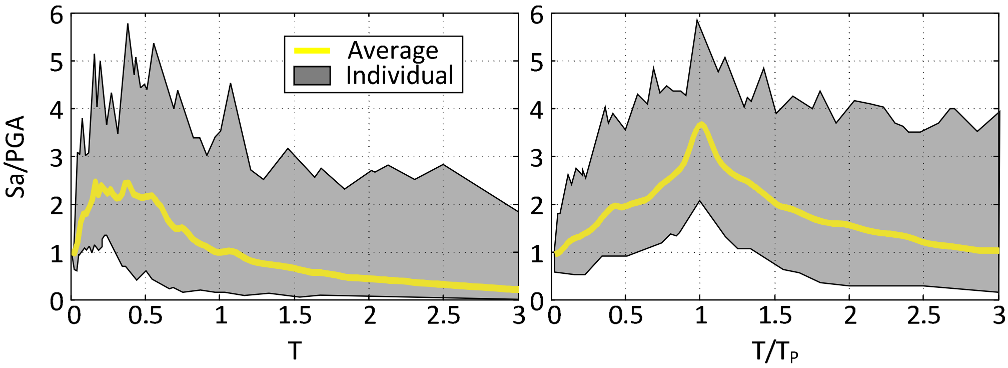

Figure 5 Bi-normalised acceleration and displacement spectra for the twenty ground motions listed in Table 1

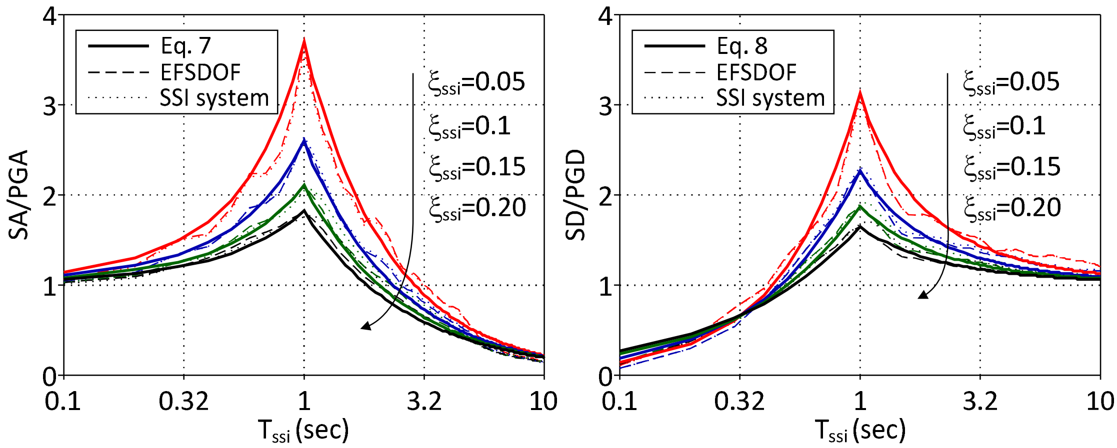

Figure 6 Displacement demands of SSI systems and their corresponding EFSDOF oscillators subjected to the 360° component of the seismic record at Larkspur Ferry Terminal in the 1989 Loma Prieta earthquake

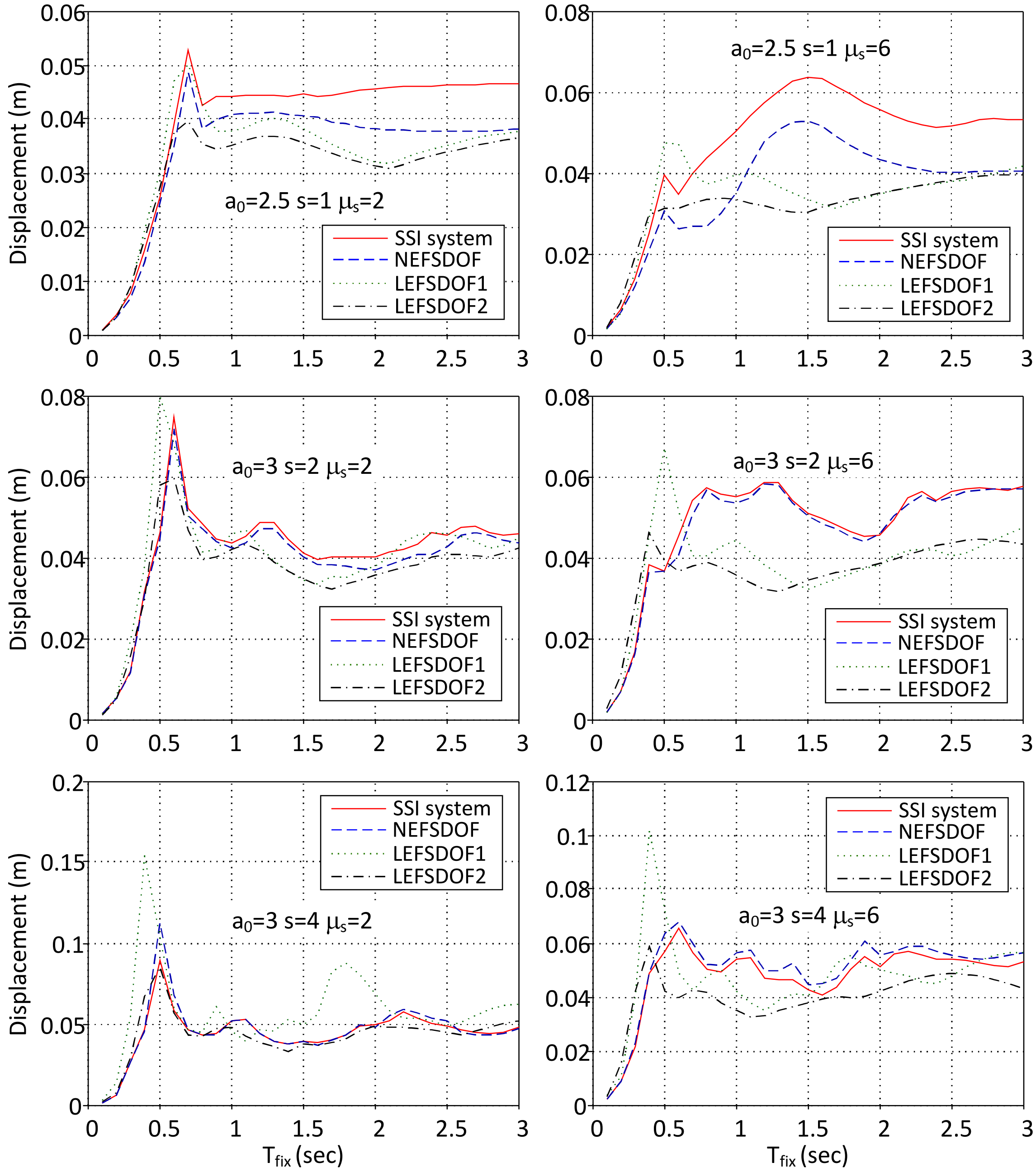

Figure 7 Comparisons of R_μ and C_μ of the SSI models having various combinations of a_0 and s with those of the NEFSDOF oscillators

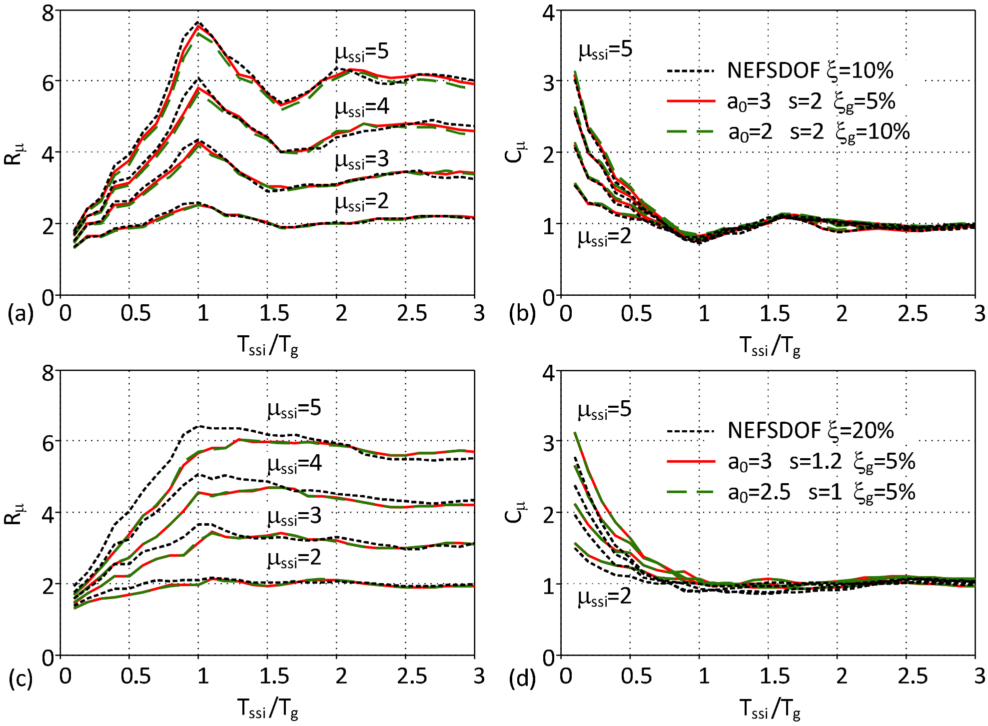

Figure 8 (a)-(b) Proposed correction factor α_ξ and (c)-(f) improved NEFSDOF oscillators for estimating R_μ and C_μ of SSI systems having an effective elastic damping ratio of ξ_{ssi}=15% and 20%

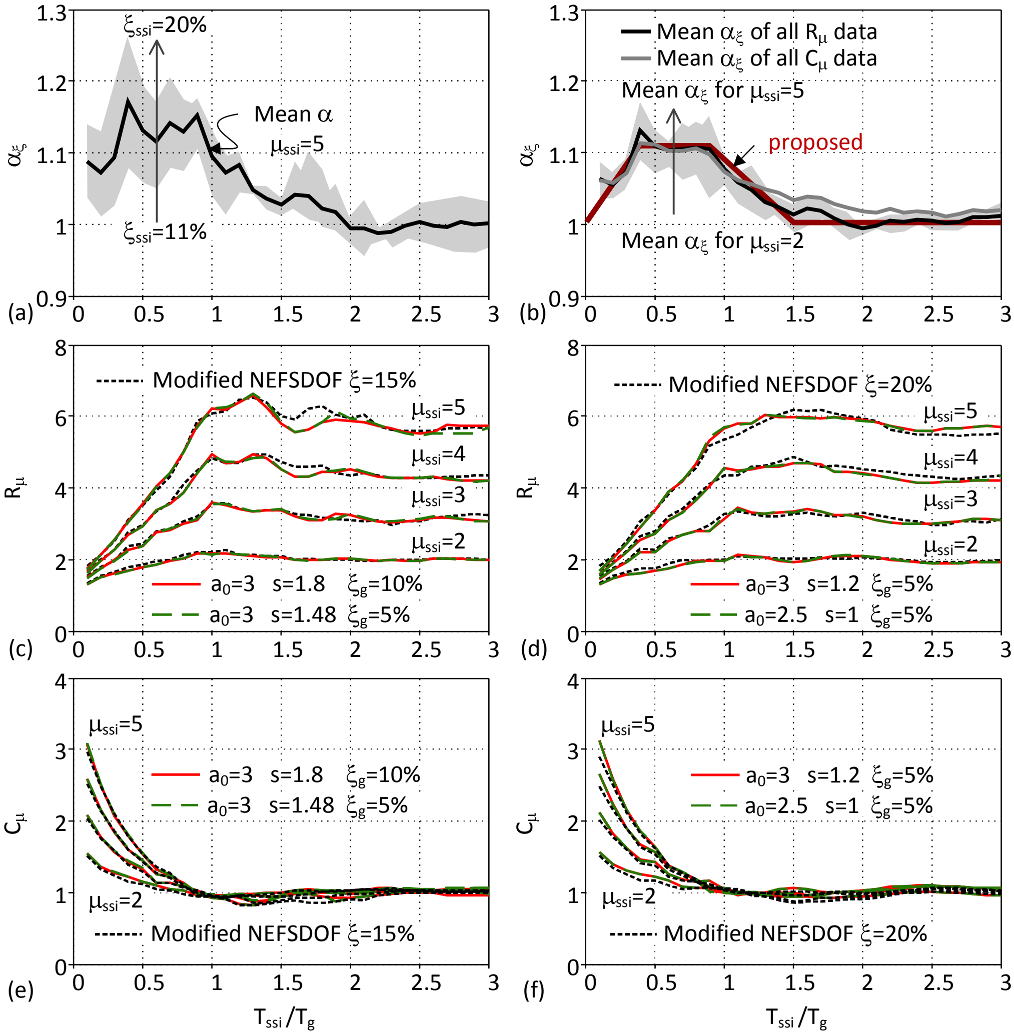

Table 1 Ground motions recorded on soft soil deposits

Index	**Event**	**Magnitude (M_s)**	**Station**	**Component (degrees)**	**PGA* (cm/s^2)**
10/17/89	Loma Prieta	7.1	Foster City (APEEL 1; Redwood Shores)	90, 360	278, 263
10/17/89	Loma Prieta	7.1	Larkspur Ferry Terminal	270, 360	135, 95
10/17/89	Loma Prieta	7.1	Redwood City (APEEL Array Stn. 2)	43, 133	270, 222
10/17/89	Loma Prieta	7.1	Treasure Island (Naval Base Fire Station)	0, 90	112, 98
10/17/89	Loma Prieta	7.1	Emeryville, 6363 Christie Ave.	260, 350	255, 210
10/17/89	Loma Prieta	7.1	San Francisco, International Airport	0, 90	232, 323
10/17/89	Loma Prieta	7.1	Oakland, Outer Harbor Wharf	35, 305	281, 266
10/17/89	Loma Prieta	7.1	Oakland, Title & Trust Bldg.	180, 270	191, 239
10/15/79	Imperial Valley	6.8	El Centro Array 3, Pine Union School	140, 230	261, 217
04/24/84	Morgan Hill	6.1	Foster City (APEEL 1; Redwood Shores)	40, 310	45, 67

***** PGA=peak ground acceleration

Table 2 Properties of the SSI systems and their corresponding EFSDOF oscillators

EFSDOF		T/T_s		ξ(%)		μ	
oscillator	SSI system	μ_s=2	μ_s=6	μ_s=2	μ_s=6	μ_s=2	μ_s=6
	a_0=2.5 s=1	1.48	1.48	20	20	1.46	3.28
NEFSDOF	a_0=3 s=2	1.82	1.82	10	10	1.30	2.51
	a_0=3 s=4	2.16	2.16	5	5	1.21	2.07
	a_0=2.5 s=1	1.75	2.14	19.9	20.3		
LESDOF1	a_0=3 s=2	2.14	2.32	10.2	13.6		
	a_0=3 s=4	2.55	2.64	2.1	6.8	N/A	
	a_0=2.5 s=1	1.79	2.68	26.9	35.2		
LESDOF2	a_0=3 s=2	2.08	2.88	14.8	22.5		
	a_0=3 s=4	2.38	3.11	8.5	15.3		