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Article

A Study on Possibility of Detrimental Effects of Soil-Structure Interaction in Seismic Design Due to a Shift of System Frequency

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Abstract: Soil-structure interaction (SSI) leads to a modification of dynamic properties of a structure, but due to the complexity on analysis, it is traditionally assumed in seismic designs that the structure is fixed-supported at the ground, which unavoidably gives potential dangers to seismic performances of the structure. This study focuses on the possibility of the detrimental effects of SSI by comparing the dynamic responses of SSI system to fixed-base structure. The SSI model consists of an oscillator, a three-dimensional foundation and a multilayered half-space. In indirect boundary element method is employed, combined with non-singular Green's functions of distributed loads to calculate the foundation impedances for substructure method. Although the SSI mechanism dampens peak amplitude of structure response in frequency domain, case studies on four buildings to 42 earthquakes in time history show a possibility of 15%-20% that SSI amplifies the dynamic responses, depending on the frequency component of incident waves.

Keywords: soil-structure interaction; indirect boundary element method; foundation impedances; system frequency; system responses; detrimental effects; beneficial effects; seismic design

1. Introduction

It is assumed in traditional seismic designs that structures are fixed-supported on the foundation. Actually, the soil performs obvious flexibility for dynamic excitations, such as earthquakes, which gives the mechanism of soil-structure interaction (SSI). Although the soil performs as a cushion under the structure foundation, scholars have come to realize that this mechanism has detrimental effects sometimes to seismic performances of structures, and ignoring this mechanism results potential risks to seismic designs [1-6]. Nevertheless, due to the complexity and the cost, seismic designs involving the SSI analysis up to now are applicable only to few important infrastructures, such as nuclear power stations and huge bridges [7-11], or to few civil structures for some special reasons [12]. Most civil structures, however, receive little attention on the seismic designs involving SSI and the fixed-base assumption is adopted for convenience.

The foundation of a structure, usually with various shapes, is a group of multi-scatters in infinite half-space, so it leads to complex scattering field under seismic excitations. Currently, the numerical simulations are still computation-consuming and technique-demanding. Actually, in the past decades, this problem has been improved much, such as raising the incident wave to a more shallow layer to reduce the range of near-field [13], setting incident waves inside the artificial boundary to reduce the data storage on free-field motion and seismic loads [14], or carrying large-scale calculation from earthquake source to structure site through multi-scale simulation [15], and so on. Nevertheless, for most civil structures, it is only possible to determine the potentially detrimental effects of SSI very roughly by the shear wave velocity, or the period, of site. The method, although widely used among civil engineers, is not accurate and cannot be parametrically quantified. For example, many studies by numerical simulations [16, 17] and earthquake damage investigations [1, 18] reveal that, structures with some same features in a certain area, exhibit much more serious damage than other structures without the features.

Certainly, there is still much work trying to integrate the SSI analysis into the framework of traditional seismic designs, for wide applications in engineering practice. In ASCE/SCI-7 [19] and NEHRP (FEMA P-1050-1/2015) [20], there are coded provisions in recommended chapters in all versions after 1990's (v1997, v2000, v2003, v2009, v2015) to involve the SSI analysis, which is based on substructure method and foundation impedance functions. In CSCE 160 (General Rules for performance-based seismic design of buildings) [21], it is also recommended similar provisions and improved the formulas on evaluation of structure modes. A research report of the National Institute of Standards and Technology (NIST) provides modified coefficient curves considering SSI based on the concept of traditional coefficient curves in seismic designs [22]. Nevertheless, these methods are rarely adopted in practice for the following two reasons. The analysis involves the calculation of foundation impedance functions, which are too complex for engineering designs. Also, the analysis considers only the beneficial effects of SSI and reduces base shear force at most 30%, which is unsafe or lack of safety reverse.

Under earthquake excitations, it is coupled between kinematic interaction and inertial interaction for SSI, so some studies focus on the former by a half-space with a foundation [23, 24], or on the latter by a structure in uniform half-space [25]. At present, there is a lack of researches considering meanwhile the two types of interactions, and the target parameters in the researches are limited [26], which is difficult to support seismic analysis in engineering. This paper investigates the possibility of detrimental effects of SSI quantitatively by comparing the dynamic responses of an SSI system to a fixed-base structure. The SSI model consists of an oscillator, a 3D rectangular foundation, and a multi-layered half-space, which considers the two interactions. Also, a feasible idea based on neural network algorithm is proposed at last, taking into account the complexity of site conditions, diversity of structure formats, and uncertainty of seismic effects, to shed some light on SSI analysis in engineering seismic designs.

2. Model and Results

The model of Soil-Structure Interaction system (SSI) consists of a superstructure, a three-dimensional rectangular foundation, and a horizontally multi-layered half-space, as shown in Figure 1. The superstructure is simplified to a single-degree-of-freedom (SDOF) oscillator, and its dynamic properties are characterized by mass M_b , spring coefficient k_b , damping coefficient c_b and height H , so the fixed-base frequency is $\omega_b = \sqrt{k_b / M_b}$ and the damping ratio is $\xi_b = c_b / 2M_b\omega_b$. The foundation is represented by a rigid rectangular foundation of mass M_0 , width a , length b and embedment c . The half-space consists of a soil layer of thickness D on a bedrock, and each sub-layer, as well as the bedrock, are assumed to be elastic and isotropic. The material properties of j th sub-layer are characterized by shear-wave velocity β_j , Poisson's ratio ν_j , mass density ρ_j and damping ratio ξ_j , and those of the bedrock are characterized by shear-wave velocity β_R , Poisson's ratio ν_R , mass density ρ_R and damping ratio ξ_R . The SSI system is excited by a train of harmonic SV wave coming from the bedrock, with motion in the XOZ-plane. The circular frequency of the excitation is ω , and the incident angle is θ measured horizontally.

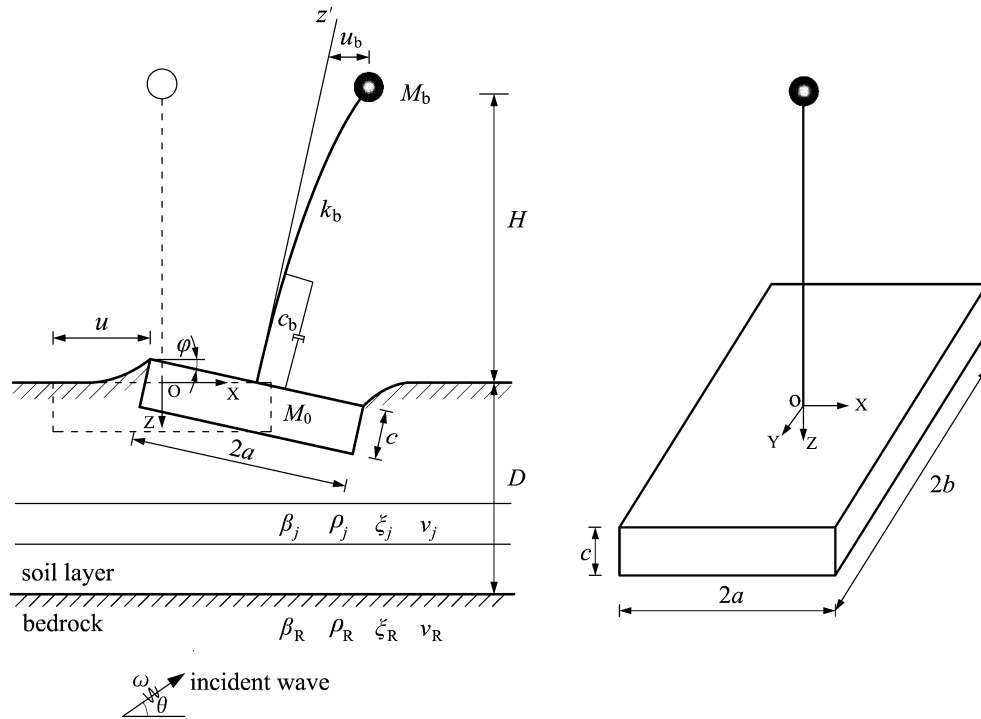


Figure 1. SSI model consisting of a superstructure, a three-dimensional rectangular foundation, and a horizontally multi-layered half-space.

The deformation of the structure is represented by the relative horizontal translation u_b with respect to z' -axis which is fixed on the foundation, and the foundation motion is represented by the generalized vector $\mathbf{U}=\{u, \varphi\}^T e^{i\omega t}$, with u of the horizontal translation and φ of the clockwise rotation about the reference point $(0, 0, c)$, and $e^{i\omega t}$ is the harmonic time factor, omitted hereafter. The vertical displacement, as well as the torsional motion, of the SSI system are not considered in our problem. The dimensionless parameters necessary in the following analysis are:

(1) The dimensionless mass of the foundation and the structure are M_0/M_s and M_b/M_0 , respectively, where M_s is the mass of soil replaced by the foundation.

(2) The dimensionless frequency is defined as $\eta=\lambda_1/2a=\omega a/\pi\beta_1$, where λ_1 is the shear wavelength of the first sublayer. Correspondingly, the dimensionless fixed-base frequency is $\eta_b=\omega_b a/\pi\beta_L$, and a general range of η_b in engineering practice is 0-0.4, and the dimensionless system frequency is $\tilde{\eta}_b=\tilde{\omega}_b a/\pi\beta_L$.

(3) The dimensionless size of the foundation and the dimensionless thickness of the soil layer are a/a , b/a , c/a and D/a , respectively.

(4) The dimensionless mass density and the shear wave velocity of the soil layer are ρ_R/ρ_j and β_R/β_j , respectively.

2.1. Dynamic Equilibrium

The dynamic equilibrium equations of the SSI are [27-29]

$$\mathbf{M}\ddot{\mathbf{V}}+\mathbf{C}\dot{\mathbf{V}}+\mathbf{K}\mathbf{V}=-\mathbf{M}\ddot{\mathbf{U}}_* \quad (1)$$

where

$$\mathbf{M} = \begin{bmatrix} M_b & M_b & M_b \\ M_b & M_0 + M_b & S_0 + M_b \\ M_b & S_0 + M_b & I_0 + I_b \end{bmatrix}, \quad \mathbf{C} = \begin{bmatrix} c_b & 0 & 0 \\ 0 & c_{hh} & c_{hm} \\ 0 & c_{mh} & c_{mm} \end{bmatrix}, \quad \mathbf{K} = \begin{bmatrix} k_b & 0 & 0 \\ 0 & k_{hh} & k_{hm} \\ 0 & k_{mh} & k_{mm} \end{bmatrix}, \quad (2)$$

$$\mathbf{M}' = \begin{bmatrix} M_b & M_b \\ M_0 + M_b & S_0 + M_b \\ S_0 + M_b & I_0 + M_b \end{bmatrix}, \quad \mathbf{V} = \begin{bmatrix} u_b \\ u_0 \\ \phi_0 H'/a \end{bmatrix}$$

Symbol S_0 is the area moment of the foundation with respect to the axis $z=c$, I_0 and I_b are the inertia moments of the foundation and the superstructure, respectively, and we have $I_b = M_b H'$ for simplicity here. That $H' = H + c$ is the structure height from the structure top to the foundation bottom. Vector $\mathbf{U}_* = \{u_*, \phi_*\}^T$ is the foundation input motion (response of the excavation where the foundation is placed) [30, 31]. Foundation impedances are complex-valued and are shown by

$$\mathbf{K}_0(\omega) = \begin{bmatrix} K_{hh}(\omega) & K_{hm}(\omega) \\ K_{mh}(\omega) & K_{mm}(\omega) \end{bmatrix} = \begin{bmatrix} k_{hh}(\omega) & k_{hm}(\omega) \\ k_{mh}(\omega) & k_{mm}(\omega) \end{bmatrix} + i\omega \begin{bmatrix} c_{hh}(\omega) & c_{hm}(\omega) \\ c_{mh}(\omega) & c_{mm}(\omega) \end{bmatrix} \quad (3)$$

The real and the imaginary parts perform as spring and dashpot coefficients of the site, with the elements K_{hh} of horizontal impedance, $K_{hm} = K_{mh}$ of coupled horizontal-rocking impedance and K_{mm} of rocking impedance. Foundation impedances are obtained by an Indirect boundary Element method (IBEM), combined with non-singular Green's functions of distributed loads presented in [31]. The foundation impedances in Equation (5) are already normalized by H'/a , to be unified to the same units.

2.2. Dynamic Properties of SSI

A simplest layered half-space, a single soil layer on bedrock, is taken for analysis on dynamic properties of SSI system. The following parameters of the site are used $\rho_R/\rho=1$, $\nu_R=\nu=1/3$, $\xi_R=\xi_f=0.02$, $\beta_R/\beta=2$, $D/a=2$, ($j=1, 2, 3, \dots$). The foundation embedment is $c/a=0.5$, and the foundation mass is $M_0/M_s=0.2$. While for the structure, we have the damping ratio $\xi_b=0.02$, the height $H/a=2$, and the mass $M_b/M_0=10$ following the recommended relationship $M_b/M_0=4.9H/a$ in [32]. The length-to-width ratio of the foundation varies as $b/a=1/4, 1/2, 1, 2$ and 4 .

Figure 2(a) shows the spectrum amplitude of structure response $|u_b|$ for a fixed-base oscillator and SSI systems with varying b/a ratio. The x -axis is the dimensionless incident frequency $\eta=0-0.3$, which covers the predominant frequency domain of energy in earthquakes. Three structure frequency is concerned $\eta_b=0.1, 0.15, 0.2$. For a fixed-base oscillator, a rigid connection between the structure and the ground is assumed without any energy dissipation through the soil, while for an SSI system, the soil behaves like a cushion under the structure and adds additional flexibility to the system, so the system frequency of SSI is obviously lower than the resonant frequency of the corresponding fixed-base oscillator. Further, as the length-to-width ratio b/a decreasing, the peak has higher frequency and higher amplitude, approaching the fixed-base case.

Figure 2(b) shows the system frequency, system damping and peak amplitude with varying b/a ratios [29, 33], but the x -axis is the dimensionless structure frequency $\eta_b=0-0.3$. Although the spectrum shape is similar between the fixed-base oscillator and the SSI system with various length-to-width ratio, the system frequency of the latter shifts up to 50% from the resonant frequency of the former. Actually, a shift of peak frequency leads to a different correspondence with respect to the frequency of excitations.

Due to the shift of peak frequency, the SSI system displays different seismic performances from the fixed-base oscillator. For a case that the predominant frequency of incident wave matches the peak frequency of SSI system (Acceleration record 1 in Figure 3), the dynamic responses of SSI system are probably larger than the fixed-base oscillator, which probably gives a detrimental effects of SSI effects on seismic performances of a structure, so the seismic designs on the assumption of fixed-base oscillator are probably unsafe. While for a case that the predominant frequency matches that of the fixed-base oscillator (Acceleration record 2 in Figure 3), the responses of the former are probably

smaller than the latter, which probably gives a beneficial effects of SSI effects, so the seismic designs on the assumption is probably too conservative. Certainly, for a case that the predominant frequency of incident wave is not obvious, it also probably results in beneficial effects of SSI effects, since the peak amplitude of SSI system is always smaller than that of the corresponding fixed-base oscillator. In summary, it is avoidable that SSI effects are sometimes detrimental to the seismic performances of structures, depending on the dynamic properties of both the structure itself and the soil.

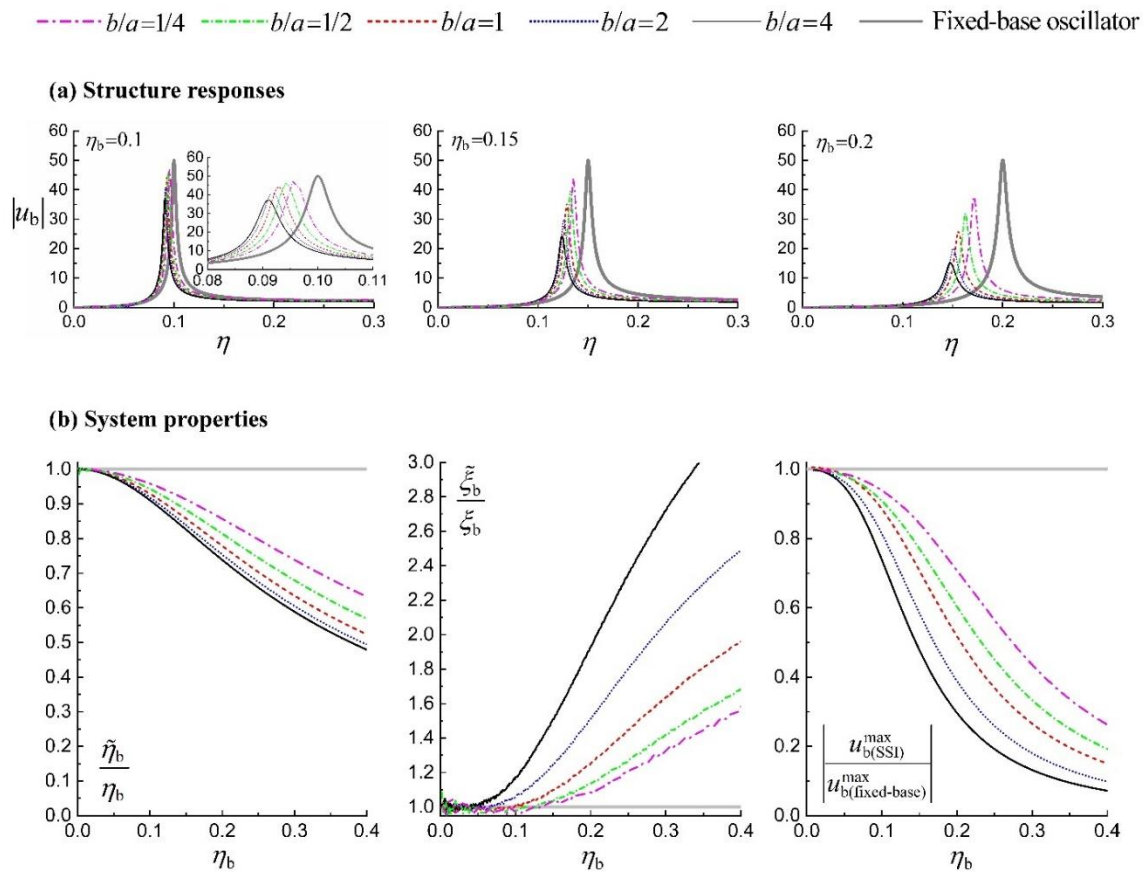


Figure 2. Dynamic responses (a) and dynamic properties (b) of SSI system ($\beta_R/\beta=2$, $D/a=2$, $c/a=0.5$, $M_b/M_0=10$, $H/a=2$).

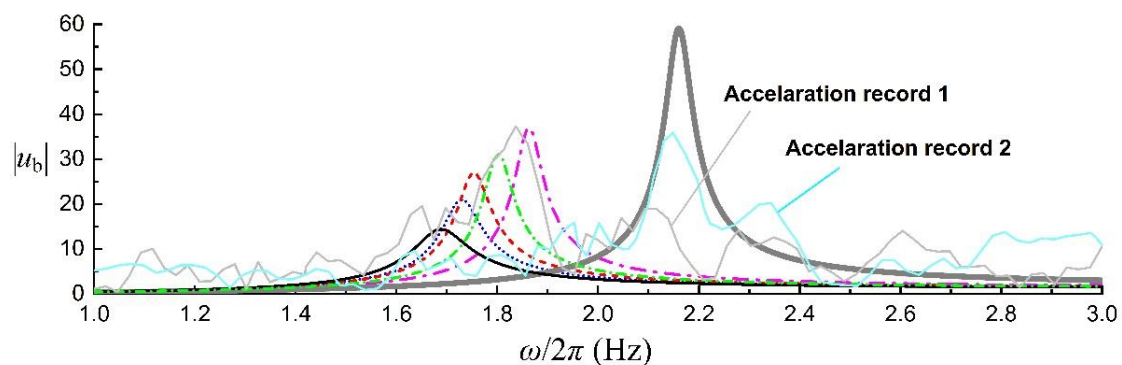


Figure 3. Peak frequency of structure response and Predominant frequency of excitations.

3. Case Studies

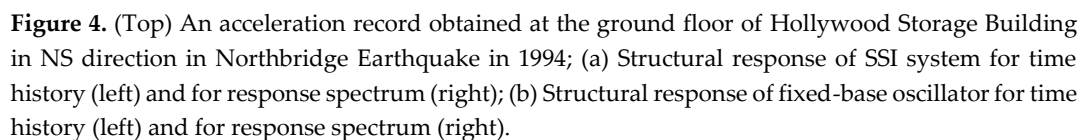
3.1. Data on Structures and Sites

A further study in time domain is performed on four buildings in California, USA, to investigate the possibility of detrimental effects of SSI. The parameters are selected from the published literature ---- Hollywood Storage Building (California Strong Motion Instrumentation Program (CSMIP) station No. 24236) [34], Millikan Library (National Strong Motion Project (NSMP) No. 5407) [35], a building in Sherman Oaks (CSMIP station No. 24322) and a building in Walnut Creek (CSMIP station No. 58364) [22]. These buildings have been instrumented for strong-motion observations for decades, and their structural and site data are available. The details of the four buildings and the stratifications of the site can be found in our previous paper [36], and we listed key information of the buildings in Table 1. The fixed-base frequency $\omega_b/2\pi$ for the four buildings is from 0.83Hz to 2.16Hz, and the shear-wave velocity of the soil at the top of the site is from 184.7m/s to 336m/s, which gives relatively soft sites with respect to the structures. It is anticipated that soft sites lead to obvious soil-structure interaction and consequently introduce much influence to structure responses.

Table 1. Buildings for case studies.

Parameters	Hollywood Storage Building	Millikan Library	Sherman Oaks Building	Walnut Creek Building
a (short side)	7.77 m	11.65 m	18.15 m	15.9 m
b (long side)	33.15 m	12.55 m	28.8 m	22.6 m
c	2.74 m + 6 m	4.3 m	6.2 m + 9.9 m	4.2 m
$\tilde{H}$	45.6 m	43.9 m	50 m	39.2 m
M_0 (kg)	1.38×10^7 (total)	1.43×10^6 kg	4.57×10^7 (total)	1.43×10^6 kg
M_b	1.17×10^7 kg	1.07×10^7 kg	1.66×10^7 kg	1.07×10^7 kg
$\tilde{\xi}_b$	2 %	2 %	2 %	2 %
XOZ	b/a	4.27	0.93	1.59
	c/a	1.12	0.34	0.89
	H/a	3.39	0.88	1.59
	$\omega_b/2\pi$	0.83 Hz	2.16 Hz	1.54 Hz
YOZ	a/b	0.23	1.08	0.63
	c/b	0.26	0.37	0.56
	H/b	0.79	2.18	1.00
	$\omega_b/2\pi$	2.0 Hz	1.26 Hz	1.69 Hz

Figure 4(a) and 4(b) are the dynamic responses of Hollywood Storage Building for an assumed SSI system and an assumed fixed-base structure, respectively. The excitation, as shown at the top, is an earthquake record obtained at the ground floor in the NS direction in Northridge Earthquake in Year 1994, and it is normalized to an acceleration of peak value 0.1 g. The record can be found from a USGS website <https://strongmotioncenter.org/>. It is also assumed that the excitation is a train of SV wave coming from the bedrock with incident angle $\theta = 90^\circ$. The acceleration responses of both the SSI system and the fixed-base structure are shown in time history $\ddot{u}_b(t)$ (left column) and response spectrum of the time history $RS(\ddot{u}_b)$ (right column).



The responses in time history in Figure 4 are conducted in the following steps. First, the time history of an earthquake acceleration was converted into the frequency domain at 4096+1 equally spaced points from 0 to 50 Hz using Fourier transformation to acquire the spectrum of earthquake acceleration. The corresponding values on the spectrum of the structure response u_b were calculated at these frequency points as well. Thereafter, the spectrum of the structure response is multiplied by the spectrum of the earthquake acceleration at the corresponding frequency points to acquire the amplified spectrum of the structure. Finally, the response in the frequency domain is converted to the time domain using inverse Fast Fourier Transformation.

Table 2. Dynamic responses in the time domain of target buildings in the XOZ-plane.

Earthquake	Year	Time History $\ddot{u}_b(t)$				$RS(\ddot{u}_b)$	
		Maximum (g)		Mean (g)×10 ⁻²		Maximum (g)	
		SSI system	Fixed-base	SSI system	Fixed-base	SSI system	Fixed-base
Hollywood Storage Building (NS)							

Kern County	1952	0.43	0.28	7.72	5.25	1.67	1.40
San Fernando	1971	0.37	0.15	5.52	3.12	1.17	0.85
Whittier Narrow	1987	0.32	0.15	4.92	4.34	0.97	0.90
Northridge	1994	0.33	0.21	4.33	2.92	0.96	0.80
Chinohills	2008	0.41	0.18	7.24	2.75	1.73	0.68
Encino	2014	0.27	0.11	1.49	0.61	0.57	0.23
Millikan Library (NS)							
Lytle Creek	1970	0.41	0.33	11.40	9.83	3.44	2.56
San Fernando	1971	0.13	0.21	3.41	5.61	0.57	1.10
Whittier Narrow	1987	0.16	0.37	3.12	7.10	0.90	2.66
Yorba Linda	2002	0.13	0.26	2.91	5.76	0.69	1.96
San Simeon	2003	0.12	0.13	3.45	3.49	0.87	1.03
Sherman Oaks Building (NS)							
Lander	1992	0.21	0.28	5.64	6.55	1.30	1.66
Whittier Narrow	1987	0.13	0.15	1.38	1.52	0.34	0.33
Northridge	1994	0.20	0.22	2.43	3.70	0.78	1.51
Chatsworth	2007	0.095	0.102	1.00	1.04	0.33	0.47
Chinohills	2008	0.20	0.16	2.44	2.57	0.73	1.01
Encino	2014	0.061	0.12	0.053	0.06	0.18	0.33
Walnut Creek Building (EW)							
Livermore	1980	0.33	0.34	4.94	6.20	1.32	1.48
Livermore Aftershock	1980	0.27	0.39	4.33	7.84	1.50	2.37
LomaPrieta	1989	0.24	0.33	4.54	5.83	1.19	1.68
Alamo	2008	0.10	0.12	1.25	1.77	0.30	0.45

Table 3. Dynamic responses in the time domain of target buildings in the YOZ-plane.

Earthquake	Year	Time History $\ddot{u}_b(t)$				$RS(\ddot{u}_b)$	
		Maximum (g)		Mean (g)×10 ⁻²		Maximum (g)	
		SSI system	Fixed-base	SSI system	Fixed-base	SSI system	Fixed-base
Hollywood Storage Building (EW)							
Kern County	1952	0.44	0.32	7.84	7.02	2.39	2.50
San Fernando	1971	0.38	0.19	4.01	3.17	1.45	1.36
Whittier Narrow	1987	0.49	0.53	8.60	15.42	2.09	4.69
Northridge	1994	0.45	0.32	6.60	4.15	3.11	1.95
Chinohills	2008	0.35	0.41	5.52	6.45	1.94	3.14
Encino	2014	0.19	0.25	2.88	3.67	0.88	1.48
Millikan Library (EW)							
Lytle Creek	1970	0.39	0.49	9.47	15.44	2.37	3.77
San Fernando	1971	0.20	0.19	6.65	7.74	1.35	1.53
Whittier Narrow	1987	0.27	0.34	5.53	6.36	1.30	1.80
Yorba Linda	2002	0.107	0.105	2.62	3.54	0.56	0.70
San Simeon	2003	0.44	0.55	11.74	14.90	3.13	4.19
Sherman Oaks Building (EW)							
Lander	1992	0.19	0.27	5.20	8.10	1.20	1.99

Whittier Narrow	1987	0.20	0.26	2.95	5.50	0.87	1.64
Northridge	1994	0.187	0.190	2.91	3.11	1.13	1.15
Chatsworth	2007	0.055	0.106	0.05999	0.0866	0.19	0.43
Chinohills	2008	0.20	0.24	3.58	5.18	1.46	1.63
Encino	2014	0.061	0.12	0.053	0.06	0.18	0.33
Walnut Creek Building (NS)							
Livermore	1980	0.28	0.37	6.25	8.17	2.07	2.42
Livermore Aftershock	1980	0.14	0.17	2.75	3.27	0.85	1.01
LomaPrieta	1989	0.14	0.20	3.09	2.94	0.96	1.33
Alamo	2008	0.13	0.19	1.11	2.15	0.36	0.59

Figure 5 illustrates the ratio of the three responses (maximum response in the time history, mean response in time history, and maximum response spectrum) of the SSI system to these of the fixed-base oscillator. The x -axis corresponds to each event in Tables 1 and 2, and y -axis is the percentage value. Symbol “FB” represents “fixed-base oscillator”. The value larger than 100% in y -axis indicates a detrimental effect of SSI in the seismic design of structures. As shown in the figure, about 15%-20% of the 42 earthquake events gives the detrimental effects, and SSI mechanism amplifies the structure responses even up to 1.5 times for few cases. It justifies that a shift on the spectrum may lead to considerably different responses in the time domain, depending on the frequency component of the incident waves, the parameters of the structure as well as the soil.

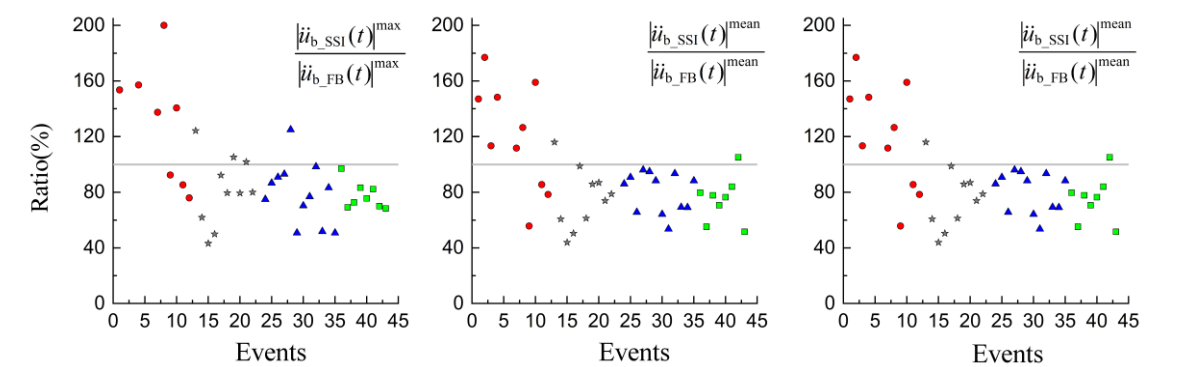


Figure 5. Ratio of dynamic responses between SSI system and Fixed-base oscillator (shown by “FB”) (Red circle: Hollywood Storage building, gray star: Millikan library, blue triangle: Sherman Oaks building, and green rectangle: Walnuts Creek building).

4. Conclusions and Discussions

Up to now, it is assumed in most seismic design works that the structures are fixed-supported on the foundation, or the underlying soil is completely rigid. Actually, the mechanism of soil-structure interaction introduces considerable differences to the seismic performances of a structure. This study investigated the possibility of the detrimental effects of SSI on the seismic performances, by comparing the dynamic responses of SSI system and fixed-base structure. The SSI model consists of an oscillator, a 3D rectangular foundation, and a multilayered half-space, and the foundation impedance functions were obtained using a high precision Indirect Boundary Element Method. Although the amplitude spectrum of SSI system is smaller than that of a fixed-base oscillator, by case studies on four buildings to 42 earthquakes, it still shows 15%-20% in possibility that SSI gives detrimental effects on seismic performances of a structure.

Seismic hazard assessment on SSI, however, is computation-demanding and time-consuming even for linear problems. For an engineering seismic analysis, efficient methods are necessary which are applicable to be involved in the standard procedures of seismic codes. Since the structure period

is the most important parameter in the seismic designs, we describe a possible method at the end of the paper to evaluate the system frequency of SSI promptly. The dynamic properties of SSI depend on the properties of the structure itself as well as the underlying soil, which are determined by a few dozens of parameters, such as the data we used in Sections 2 and 3. So a neural network function mapping the parameters of the two substructures (structure and soil) to SSI system is feasible. First, the soil that determines the dynamic characteristics of SSI is the surface layer of dozens or hundreds of meters, the constructions and the parameters of which are statistically consistent in adjacent areas of the same geological environment. Second, the structure parameters follow general principles in structural designs and engineering practices.

Actually, SSI system is multi-parametric. By traditional studies on Dynamics, it needs orthogonal comparisons on dozens of target parameters, which gives considerably large workloads, and it is difficult to obtain the global function mapping parameters of subdomains to the dynamic properties of SSI system. Neural network function is an algorithm that fits complex mapping process of multiple inputs and outputs, for which, it is difficult to obtain explicit functions. Through appropriate settings on network structure and machine learning, neural network algorithm can converge to any function theoretically. The method is applicable to cities whose geological stratum data is plentifully accumulated in the past decades, to obtain a numerical model on seismic hazard assessment involving SSI effects. This method is our future work. It should be noted that both the soil and structure are likely to experience some degree of nonlinearity during strong earthquake events, and the neural network method under nonlinear conditions are also our future work.

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