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Influence of foundation flexibility on the seismic response of low-to-mid-rise moment-resisting frame buildings

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KEYWORDS

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Nonlinear analysis;
Design code.

Abstract. Recent studies have shown that the effects of Soil-Structure Interaction (SSI) may be detrimental to the seismic response of structure, and the neglect of SSI in analysis may lead to an un-conservative design. The objective of this study is to simulate the performance of multi-storey building-foundation systems through a Winkler-based approach. Four typical steel Moment-Resisting Frame (MRF) buildings on three soil types with shear wave velocities of less than 600 m/s subjected to actual ground motion records of varied hazard levels are modeled with and without SSI. It is observed that the performance level of models supported by flexible foundation, particularly in an intense earthquake event, may alter significantly in comparison to fixed-based structures. Moreover, for MRFs on soft soil, the nonlinear foundation is found to have a significant effect on the force and displacement demands. This indicates the necessity for consideration of flexible foundation behavior in the modern design codes in order to accomplish a more economic yet safe structural design.

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1. Introduction

The dynamic response of a structure to earthquake excitation can significantly be affected by interactions between three linked systems: the structure, the foundation, and the soil underlying and surrounding the foundation. In reality, the supporting soil somewhat influences the structural response by permitting movement due to its natural ability to deform. Soil-Structure Interaction (SSI) analysis evaluates the collective response of these systems to a specified free-field ground motion. When the ground is stiff enough, the dynamic response of the structure will not be influenced significantly by the soil properties during

the earthquake, and the structure can be analyzed under the fixed-base condition. But, for the structure resting on a flexible medium, the dynamic response of the structure will be different from that of the fixed-base condition, owing to the interaction between the soil and the structure [1]. In the case of a flexible-base structure, in addition to the added rocking component to the horizontal motion of the structure, a part of the structure's vibrating energy will be transmitted to the soil layer, and it can be dissipated due to radiation damping resulting from the wave propagation and hysteresis damping of the soil materials.

In general, SSI effects can be summarized as follows: reduction of the natural frequency of the system, increase in damping, increase in the rate of the lateral displacement, and change in the force demands of the structure [2,3]. For stiff structural systems, such as shear wall and braced frame founded on soil, ignoring the influence of foundation movements could

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lead to significant misestimation of the fundamental frequency in the system [4-6]. Analyses conducted under various soil and structure conditions showed that this influence depends mainly on the soil-structure relative rigidity. The increased period and damping of the soil-structure system largely lead to the consequences and the beneficial effects the SSI has, which are mostly ignored in seismic design of buildings. This conclusion could be misleading, since ignoring base flexibility may over-or under-predict seismic response of the structure depending primarily on the characteristics of the ground motions. It was shown that the SSI can play a detrimental role in producing such influences, and neglecting its influence could lead to an unsafe design [7,8]. According to more recent findings, neglecting the effects of SSI on seismic performance of rocking structures with shallow foundations may drastically bias the estimation of force demands against near-fault pulse, such as ground motions [9]. Therefore, it is recommended to take the nonlinear SSI effects into consideration in order to avoid rough estimation of structural seismic demands.

Although not widely used in practice, engineering guidelines are available for simple evaluation of SSI effects. Recent code-compliant seismic designs for SSI systems, such as NEHRP [10] and ASCE 7 [11], are based on the approximation to which the predominant period and associated damping of the corresponding fixed-base system are modified. On the other hand, ATC 40 [12] and FEMA 356 [13] partially address the effects of flexible foundation through inclusion of the stiffness and strength of the soil components of the foundation (Winkler-based model) on the structural analysis. However, none of these procedures addresses the shaking demand on the structure relative to the free-field motion caused by kinematic interaction or the foundation damping effect. Guidelines on the inclusion of the effects of kinematic interaction are given in FEMA 440 [14] and ASCE 41 [15]. The Ratio of Response Spectra (RRS) factor is used to represent kinematic interaction effects in these codes. RRS is simply the ratio of the response spectral ordinates imposed on the foundation (i.e., the foundation input motion) to the free-field spectral ordinates. It is noteworthy to mention that the kinematic effects are usually most pronounced in short periods, and hence are unlikely to significantly affect responses of buildings within a fundamental period greater than approximately 1 sec [6,16].

SSI, particularly for superstructures resting on relatively soft soils, may significantly amplify the lateral displacements and inter-storey drifts [17,18]. The amplification of lateral deformations may change the performance level of the building frames while subjected to earthquakes. Tabatabaiefar et al. [18] investigated the accuracy of nonlinear method against

equivalent linear method for dynamic analysis of SSI, using shaking table tests. They concluded that the results obtained from the fully nonlinear method of analysis fit the experimental results reasonably well, but the equivalent linear method underestimates the inelastic seismic response of mid-rise moment-resisting building frames resting on soft soils. Therefore, the fully nonlinear method was recommended by practicing engineers to be used.

Various numerical methods were used to model the behaviour of structures on shallow foundations [19-22]. However, the application of simple methods, such as the Winkler approach, is preferred in practical SSI problems. In this context, Beam-on-Nonlinear-Winkler-Foundation (BNWF) method, proposed by Harden et al. [23], Harden and Hutchinson [24], and later by Gajan et al. [25], has been widely applied in recent studies due to its relative simplicity and minimal computational effort (e.g. [9,26,27]). The present article attempts to cover a wide range of interaction problems in terms of the superstructure, the soil characteristics, and the seismic excitation intensities in which the BNWF method is used to model the behavior of shallow foundation. To do so, an extensive parametric study, including different approaches of analysis along with various soil and structure conditions, is carried out to evaluate seismic response of low-to-mid-rise steel Moment Resisting Frame (MRF) buildings subjected to earthquake motions of different hazard levels. The numerical results indicate that the SSI which is significantly important to the performance-based design of structures can alter the force and drift demands.

2. Structural and geotechnical description of the models

A set of 4-, 8-, 12-, and 16-storey steel MRFs located on hypothetically soft, medium, and hard sites is considered. The building models have a floor plan of 18.0×18.0 m, three bays each having a horizontal direction at intervals of 6.0 m, a uniform mass distribution over their height, and a non-uniform stiffness distribution (Figure 1). The storey height of the models was considered as 3.3 m; a normal height for residential buildings. These models represent a large number of conventional low-to-mid-rise buildings in mega cities at relatively high-risk earthquake-prone countries, such as Iran. Buildings were designed as special frames based on the requirement of Iranian national building codes [28,29] and Iranian code of practice for seismic resistant design of buildings (Standard No. 2800) [30]. The dead and live loads of 600 and 200 kgf/m², respectively, were used for gravity loads. Consequently, the self-weight of 306720, 643960, 982530, and 1334880 kg were obtained for 4-, 8-, 12-, and 16-storey frames,

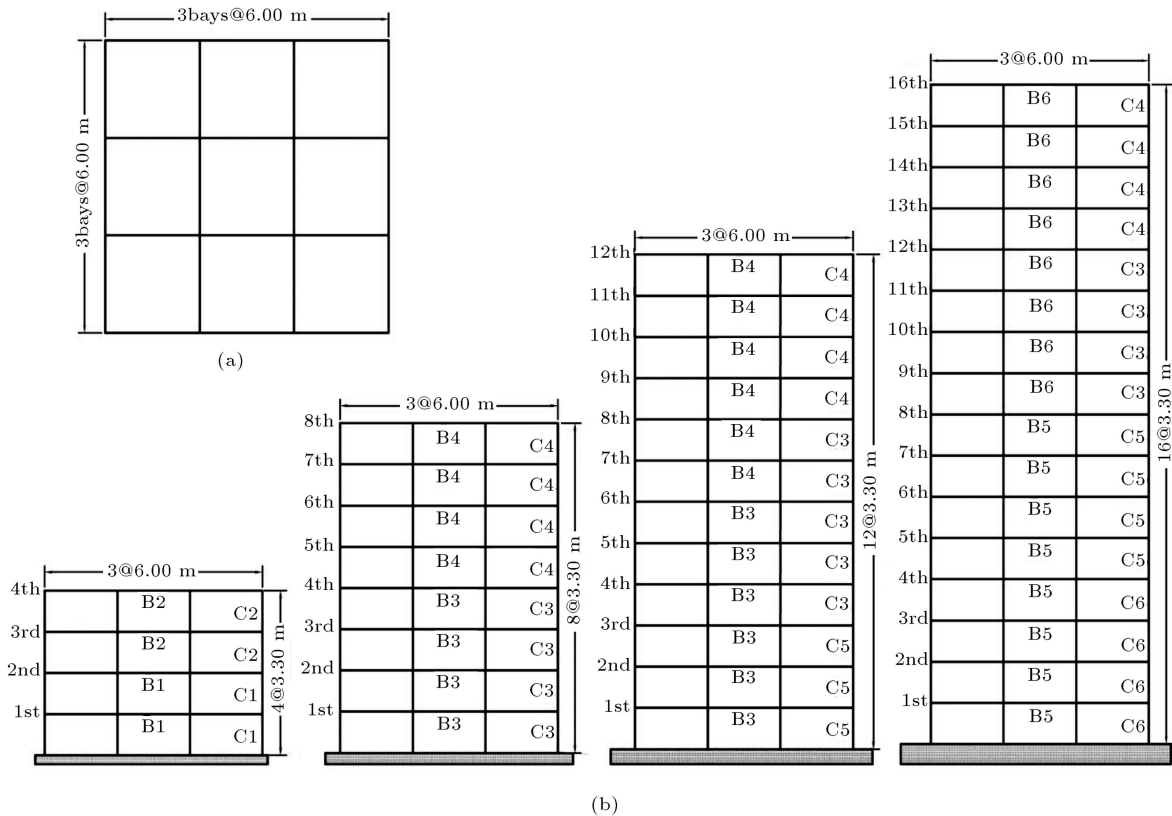


Figure 1. Configuration of building models: (a) Plan and (b) elevation.

respectively. In regard to the frames' design, the importance factor of $I = 1$, response modification factor of $R = 7.5$, and seismic zone factor of $A = 0.30$ were considered. In this paper, the steel type St-37 was used for the structural beam and column members. Table 1 lists the details of utilized I-beam sections for the MRF models shown in Figure 1.

Columns are supported by strip footings which rest on different soil conditions including types II ($375 < V_s < 750$ m/s), III ($175 < V_s < 375$ m/s), and IV ($V_s < 175$ m/s) according to the classification of the Standard No. 2800. It is worthy to note that for

soils with shear wave velocity greater than 600 m/s, the effect of SSI is not considerable [20,31]. The details of various soil parameters are as those tabulated in Table 2, extracted from the actual geotechnical projects [32,33]. The material properties used in the reinforced concrete foundation design are modulus of elasticity $E_c = 2.19 \times 10^5$ kgf/cm², compressive strength $f_c = 210$ kgf/cm², Poisson ratio $\nu = 0.2$ for concrete, and yield strength $f_y = 4000$ kgf/cm² for steel. Table 3 summarizes the characteristics of the footings in this study, where q_{all} is allowable bearing capacity of soil, K_s is the subgrade soil reaction.

Table 1. Details of steel I-beam sections.

Section tag	Web		Flange	
	Depth (cm)	Thickness (cm)	Width (cm)	Thickness (cm)
C1	31.95	2.11	39.88	3.33
C2	32.10	1.64	37.34	2.62
C3	44.40	2.00	50.00	2.80
C4	35.60	2.00	40.00	2.20
C5	53.40	2.50	60.00	3.30
C6	62.00	3.00	70.00	4.00
B1	27.84	1.09	30.48	1.70
B2	27.65	0.99	30.48	1.54
B3	38.18	1.49	26.42	2.50
B4	38.23	1.33	26.42	2.22
B5	50.09	1.16	21.08	1.88
B6	50.11	1.09	21.01	1.74

Table 2. Details of soil parameters.

Soil type	ϕ (degree)	C (kgf/cm ²)	v	γ (kgf/m ³)	G (kgf/cm ²)	V_s (m/sec)
II	30	0.15	0.35	2100	6707	560
III	27	0.10	0.40	1900	732.2	275
IV	15	0.03	0.40	1700	339.3	150

Table 3. Foundation characteristics used in analyses.

Building model	Soil type	Footing type	B (m)	L (m)	H (m)	q_{all} (kgf/cm ²)	K_s (kgf/cm ³)
4-storey	II	Strip	1.00	19.00	0.65	2.00	2.40
	III	Strip	1.20	19.20	0.65	1.50	2.00
	IV	Strip	1.50	19.50	0.65	1.00	1.50
8-storey	II	Strip	1.70	19.70	0.90	2.00	2.40
	III	Strip	2.20	20.20	0.90	1.50	2.00
	IV	Strip	2.80	20.80	0.90	1.00	1.50
12-storey	II	Strip	2.60	20.60	1.20	2.00	2.40
	III	Strip	3.30	21.30	1.20	1.50	2.00
	IV	Strip	4.00	22.00	1.20	1.00	1.50
16-storey	II	Strip	3.50	21.50	1.50	2.00	2.40
	III	Strip	4.50	22.50	1.50	1.50	2.00
	IV	Strip	5.00	23.00	1.50	1.00	1.50

Table 4. Ground motions selected in the present study [34].

No.	Earthquake	Year	Station	M_w^a	Soil type	d^b (km)	PGA ^c (g)	PGV ^d (cm/s)	PGD ^e (cm)	T_0^f (sec)
1	Northridge, USA	1994	Old Ridge Route	6.7	II	22.6	0.57	52.1	4.2	0.26
2	Cape Mendocino, USA	1992	Rio Dell Overpass	7.1	II	18.5	0.55	42.1	18.6	0.42
3	Chi-Chi, Taiwan	1999	TCU045	7.6	II	24.0	0.51	39.0	14.3	0.40
4	Loma Prieta, USA	1989	Gilroy Gavilan Coll.	6.9	II	12.0	0.36	28.6	6.3	0.40
5	San Fernando, USA	1971	Lake Hughes	6.6	II	20.3	0.37	17.0	1.6	0.16
6	Victoria, Mexico	1980	Cerro Prieto	6.1	II	17.0	0.62	31.6	13.2	0.06
7	Whittier Narrows, USA	1987	LA-116th St School	6.0	II	22.5	0.39	21.0	1.8	0.14
8	Chuetsu-Oki, Japan	2007	Kashiwazaki NPP	6.8	III	11.0	0.45	125	49.8	1.90
9	El Mayor-Cucapah, USA	2010	RIITO	7.2	III	13.7	0.39	52.4	50.5	0.12
10	El Mayor-Cucapah, USA	1987	Cerro Prieto	7.2	III	11.0	0.29	49.5	40.8	0.44
11	El Mayor-Cucapah, USA	1987	Michoacan	7.2	III	16.0	0.54	61.6	34.6	0.24
12	Loma Prieta, USA	1989	Gilroy Array #4	6.9	III	14.5	0.42	40.2	8	0.44
13	Morgan Hill, USA	1984	Gilroy Array #4	6.2	III	12.0	0.35	17.3	3.4	0.24
14	Northwest China	1997	Jiashi	6.1	III	18.0	0.3	19.3	3.1	0.20
15	Darfield, New Zealand	2010	Cristchurch Res.	7.0	IV	19.5	0.27	62.2	54.9	0.54
16	Iwate, Japan	2008	MYG006	6.9	IV	30.4	0.24	41	24	0.72
17	Tottori, Japan	2000	SMN002	6.	IV	16.6	0.18	22	12	0.98
18	Loma Prieta, USA	1989	Redwood City	6.9	IV	48.0	0.28	53.6	12.7	1.06
19	Loma Prieta, USA	1989	Foster City	6.9	IV	43.0	0.30	37.2	12.0	0.66
20	Superstition Hills, USA	1987	EL Centro Imp.	6.5	IV	18.2	0.36	45.0	18.1	0.22
21	Westmorland, USA	1981	Westmorland Fire	5.9	IV	11.0	0.37	41.0	11.0	0.59

^a M_w : Moment magnitude; ^b d : Closest distance to fault; ^cPGA: Peak Ground Acceleration; ^dPGV: Peak Ground Velocity;

^ePGD: Peak Ground Displacement; ^f T_0 : Predominant period.

B , L , and H are the width, length, and thickness of foundation, respectively. Factor of safety against vertical bearing failure in the absence of lateral loads was also considered nearly three, which is reasonable for realistic structures.

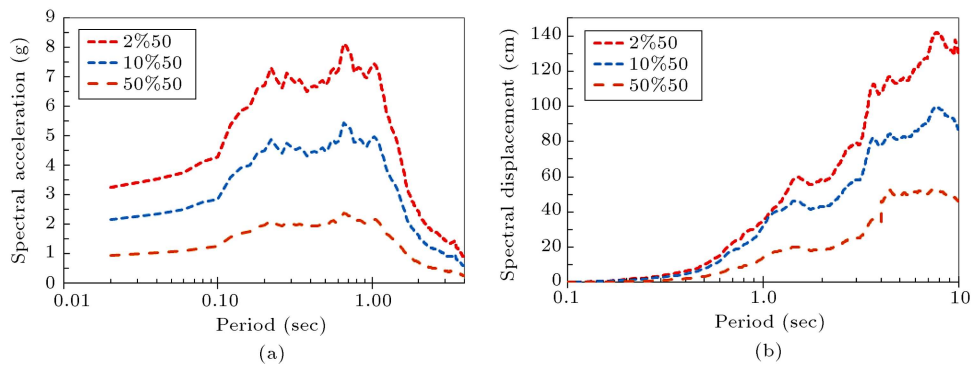
3. Earthquake ground motions

A database of 21 recorded ground motion time histories

with a wide range of intensity, duration, frequency contents, and earthquake magnitudes (i.e., M_w 5.9–7.6) was compiled from well-known studied seismic events. All motions, recorded at closest fault distances greater than 10 km, are divided into three groups of stiff, medium, and soft soil sites. Each soil type is represented by an ensemble of seven ground motions taken from the PEER NGA database [34]. Table 4 lists the characteristics of interest for the selected site

Table 5. Scale factors computed for four buildings and for three sets of seven ground motions.

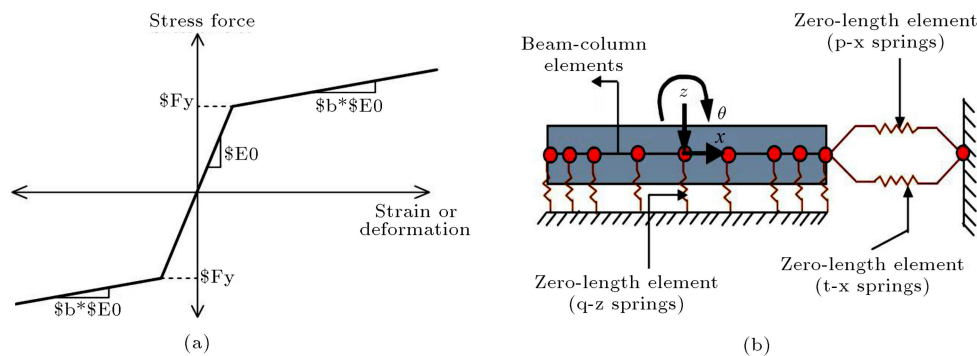
No.	Hazard level	Soil type	Scale factor			
			4-storey	8-storey	12-storey	16-storey
1	50% in 50 years	II	0.70	0.57	0.66	0.95
2	10% in 50 years	II	1.61	1.31	1.51	2.18
3	2% in 50 years	II	2.41	1.96	2.27	3.27
4	50% in 50 years	III	0.89	0.89	0.89	0.89
5	10% in 50 years	III	2.04	2.04	2.04	2.04
6	2% in 50 years	III	3.06	3.06	3.06	3.06
7	50% in 50 years	IV	0.64	0.60	0.94	1.17
8	10% in 50 years	IV	1.47	1.38	2.15	2.67
9	2% in 50 years	IV	2.21	2.07	3.22	4.01

**Figure 2.** (a) Mean acceleration response spectra, and (b) mean displacement response spectra for 5% damped elastic systems. Comparisons are plotted for three hazard levels at soil type IV scaled for 12-storey model.

and structure-specific ground motion records. More details of these motions can be found in [35]. The earthquake records were separately scaled according to the procedures described in Iranian Standard No. 2800 at seismic hazard levels of probability of exceedance of 50%, 10%, and 2% in 50 years. Table 5 lists the scaled factors computed for each set of seven ground motions of each building. For instance, as shown in Figure 2, there is the 5% damped mean response spectra for the three hazard levels at site classification IV scaled for 12-storey model. Figure 2(a) shows the spectral acceleration; Figure 2(b) shows the spectral displacement.

4. Numerical analysis

The computational model of the soil-foundation-structure systems was developed by using the OpenSees finite element software [36]. The structural members, beams, and columns were modelled as nonlinear beam-column elements using the material behaviour of steel01 with a kinematic strain hardening of 3% (Figure 3(a)). The used section for each member is the fiber section. The development of realistic numerical model of the foundation with its supporting subgrade soil was recognized as an important and

**Figure 3.** (a) Steel01 material for MRFs. (b) The schematic diagram of BNWF (after [36]).

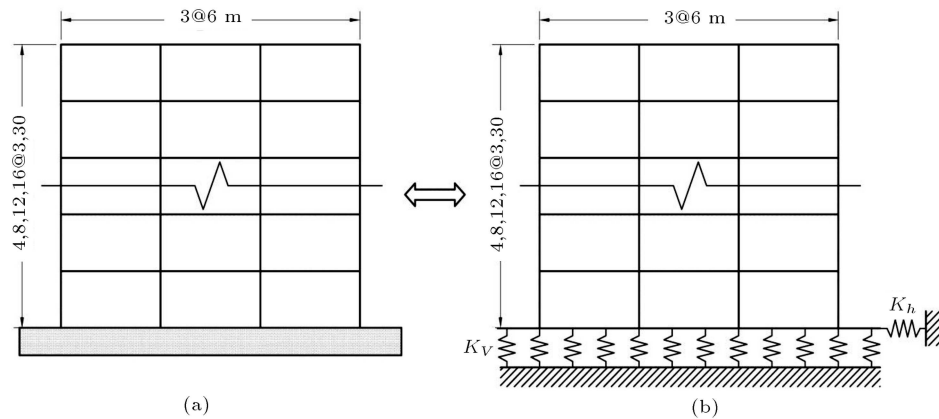


Figure 4. Different base conditions considered in the study: (a) Fixed base, and (b) nonlinear Winkler-based SSI model.

complex problem in earthquake engineering. Allotey and Naggar [37] elaborated on a Winkler-based approach utilizing multi-linear and no-tension backbone curves. Also, Harden and Hutchinson [24] developed a Winkler-based method using pile-calibrated nonlinear backbone curves of Boulanger et al. [38] to model shallow strip foundations, which can capture its rocking behaviour. The corresponding Beam-on-Nonlinear-Winkler-Foundation model was updated based on shallow footing load tests by Raychowdhury and Hutchinson [39] and implemented into the framework of OpenSees, as shown in Figure 3(b).

In this paper, the nonlinear response of MRF buildings is studied considering two comparative base conditions. First, rigid base condition means that the foundation system is assumed to be fixed in confronting all the movements in NSSI model (Figure 4(a)). The second is the SSI case in which foundation uplifting and soil plasticity are included, and the soil-foundation interface is modelled as nonlinear Winkler springs (Figure 4(b)). Vertical and lateral stiffness cases are selected based on recommendations by Gazetas [40], given in the following equations:

$$k_v = \frac{GL}{1-v} \left[0.73 + 1.54 \left(\frac{B}{L} \right)^{0.75} \right], \quad (1)$$

$$k_h = \frac{GL}{2-v} \left[2 + 2.5 \left(\frac{B}{L} \right)^{0.85} \right], \quad (2)$$

where k_v and k_h are the vertical and lateral initial stiffness of the spring, respectively. As indicated in Section 2, G is the shear modulus of the soil, v is the Poisson's ratio of the soil, and B and L are the footing width and length, respectively. Vertical load bearing capacity was calculated after Terzaghi [41], while using foundation shape and depth factors was proposed by Meyerhof [42]. Tension capacity of the soil was assumed to be 10% of the compression capacity, q_{ult} , for soil modeling. The stiffness intensity and the

spring spacing along the footing length were chosen based on the study of Harden and Hutchinson [24]. A numerical study was carried out to evaluate the design parameters' effects on the buildings seismic demands, such as fundamental period, storey deflection, storey drift, base shear, and inter-storey shear. The design parameters include the SSI with three soil types and three hazard levels based on Iranian seismic code.

5. Results and discussion

In this section, the effects of SSI are comprehensively analyzed for multiple hazard levels on soil types II, III, and IV. OpenSees platform is adopted to perform an eigenvalue analysis, nonlinear static pushover, and nonlinear time history analysis. The results of different cases are extracted, compared, and discussed as follows.

5.1. Eigenvalue analysis

The vibration period is an important parameter of a structure to estimate its seismic demand. Modern building codes generally use the period ratio (flexible-base period, T , to fixed-base period, T_{FB}) of buildings to assess their response to seismic loadings. Table 6 provides the vibration periods of the building models with different base conditions. Since the results of the eigenvalue analysis indicate that the fundamental periods computed from rigid base models are shorter than those of SSI models, it becomes insignificant for the higher modes. As shown in Table 6, fundamental period ratio is higher than 1.0, ranging from 1.02 to 1.07 for 4-storey model; from 1.01 to 1.07 for 8-storey model; from 1.01 to 1.06 for 12-storey model; and from 1.01 to 1.06 for 16-storey model. Note that as the soil stiffness decreases, fundamental period response ratio increases. It is also observed that the buildings' height has little effect on the period ratio, and can thus be neglected in evaluating the eigenvalue properties of the system. In general, Table 6 reveals that for given MRF buildings, the fundamental frequency of the soil-foundation-structure system is close to that of a fixed-

Table 6. Vibration periods of the steel MRF models with different base conditions.

Mode	4-storey				8-storey				12-storey				16-storey			
	T_{FB}^a (sec)	PR-II ^b	PR-III ^c	PR-IV ^d	T_{FB} (sec)	PR-II	PR-III	PR-IV	T_{FB} (sec)	PR-II	PR-III	PR-IV	T_{FB} (sec)	PR-II	PR-III	PR-IV
1	0.78	1.02	1.05	1.07	1.20	1.01	1.04	1.07	1.76	1.01	1.03	1.06	2.33	1.01	1.03	1.06
2	0.23	1.02	1.03	1.03	0.41	1.00	1.02	1.02	0.60	1.01	1.02	1.02	0.77	1.00	1.01	1.03
3	0.11	1.02	1.03	1.03	0.23	1.00	1.00	1.0	0.33	1.00	1.01	1.01	0.43	1.00	1.00	1.01

^a T_{FB} : Fixed-base period of building;^bPR-II: Period ratio for flexible-base to fixed-base building on soil type II;^cPR-III: Period ratio for flexible-base to fixed-base building on soil type III;^dPR-IV: Period ratio for flexible-base to fixed-base building on soil type IV.

base structure. Nevertheless, the flexibility introduced by the soil-foundation system will play an important role in altering the overall force and displacement demand of the buildings, as will be demonstrated later in this paper.

5.2. Pushover analysis

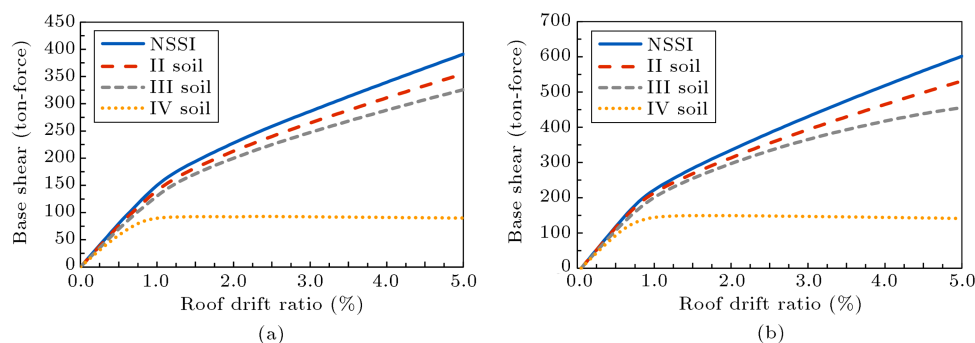
Nonlinear static analysis using pushover procedures is widely carried out in the current engineering design practice to predict the inelastic force-deformation behavior of the structure. To perform the pushover analysis, lateral loads in the form of the recommended patterns of FEMA 356 [13] were applied to the building models, followed by a displacement-controlled pushover analysis to determine the base shear-roof displacement variation for different base conditions. Figure 5 shows base shear versus the roof drift ratio (roof displacement divided by the roof elevation) for 4- and 8-storey frames. As shown in Figure 5, the SSI models present softer behavior (i.e., decrease of yield force and drift demand), indicating that the springs at the foundation-soil interface are yielding, and as a result, they modify the global stiffness of the systems. It is also observed that by decreasing the rigidity of soil, the difference between pushover curves in the fixed-base and the SSI models will be increased. The disparity in pushover curves with decreasing soil rigidity indicates that the structure-foundation system is getting softer due to the capacity mobilization of a larger number of springs. Note that the detailed seismic performance assessment of MRFs through the static nonlinear analysis by

taking the SSI into account is a matter of further research. However, the objective of the present study is to provide a simple yet useful understanding of the expected behavior of the examined structures under pushover analysis.

5.3. Time history analysis

Following the pushover analysis, nonlinear dynamic time history analyses are carried out using the ground motion records in Table 4. Newmark linear acceleration method is used to conduct the transient analysis with solution parameters of $\gamma = 0.5$ and $\beta = 0.25$. A Rayleigh damping of 5% is assumed for the frame vibrating in its first and third modes as typically considered in literature for steel frame structures. Also, the modified Newton-Raphson algorithm is used, with a convergence tolerance of $1.0\text{E-}8$ over a maximum of 60 iterations, to solve the nonlinear equilibrium equations. The transformation method, which transforms the stiffness matrix by condensing out the constrained degrees of freedom, is used in the analysis as a constraint handler. This method reduces the size of the system for multi-point constraints [36].

A total of 504 time history analyses was performed (four MRF models, twenty one ground motion records, three hazard levels, and two types of base conditions). They capture both geometrical (uplift, sliding, and rocking motions) and material nonlinearities. A number of representative time history results are selected for the purpose of studying the general trend of SSI effects, as shown from Figures 6 to 16. The

**Figure 5.** Base shear vs. roof drift ratio, with and without SSI: (a) 4-storey frame, and (b) 8-storey frame.

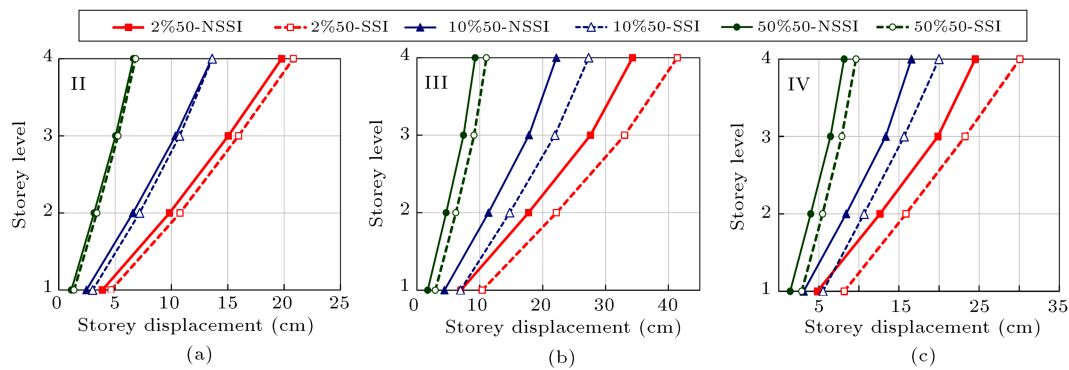


Figure 6. Storey deflection distributions of the 4-storey model for the three hazard levels at site classification: (a) II, (b) III, and (c) IV.

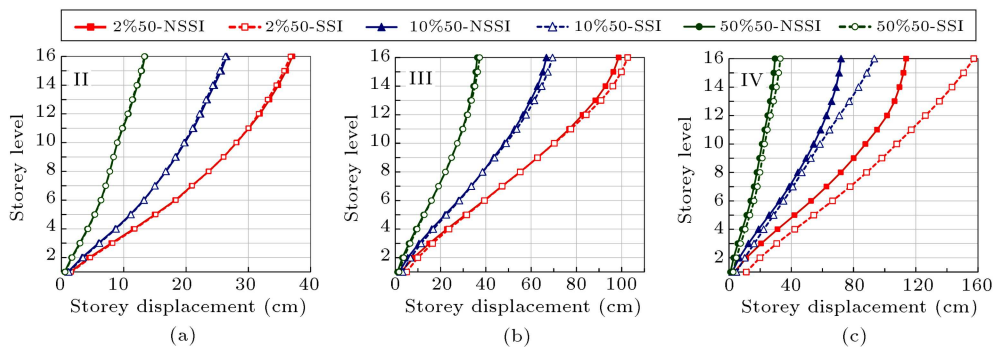


Figure 7. Storey deflection distributions of the 16-storey model for the three hazard levels at site classification: (a) II, (b) III, and (c) IV.

storey deflection, storey drift, base shear, and inter-storey shear force are chosen as the most important response parameters to estimate the seismic vulnerability of a structure in design practice. For convenience, response ratio is also defined as: (peak response of structure in SSI model) / (peak response of structure in fixed-base model). The following subsections discuss the effects of subsoil rigidity and seismic intensity on inelastic seismic response of the moment-resisting frames and their performance levels. In order to reach a systematic criterion to analyze SSI effects, the average responses for each hazard level were evaluated and presented.

5.3.1. Storey deflection

The objective of inelastic seismic analysis method is to estimate directly and accurately the magnitude of inelastic structural deformations. The average peak absolute displacements at the floor level in the direction of applied acceleration for 4- and 16-storey frames are depicted over buildings' height in Figures 6 and 7. It is observed that the storey displacement increases in SSI models, the displacement increases more in foundations located on soft soil, and this value is the largest one for the highest intensity motion (2% in 50 years). As shown, storey displacement profile increases nonlinearly with the structural height. The

increase in storey deflection occurs due to the overall reduction in the global stiffness resulting from the induced foundation movements. This trend of increase in displacement demand may be expected considering the displacement response spectra (see Figure 2(b)).

Figure 8(a)-(c) shows the storey deflection ratio distribution for the representative 12-storey frame. The storey deflection increases over all storeys in the SSI model, as seen in Figure 8. However, the rate of increase becomes higher for the 1st and 2nd storeys. It could be observed that the average response ratios of SSI-II, SSI-III, and SSI-IV reach the rates presented as follows: for 50% in 50 years hazard 1.11, 1.75, and 2.78, respectively; for 10% in 50 years hazard 1.12, 1.79, and 2.39, respectively; for 2% in 50 years hazard 1.12, 1.76, and 2.46, respectively. It is necessary to note that lower storeys are more affected by SSI than the other storeys.

5.3.2. Storey drift

Storey drift ratio is the maximum relative displacement of each floor divided by the height of the same floor and is the most commonly used damage parameter. On the other hand, seismic performance is described by considering the maximum allowable damage performance for an identified seismic hazard. Having been subjected to a certain hazard, performance levels

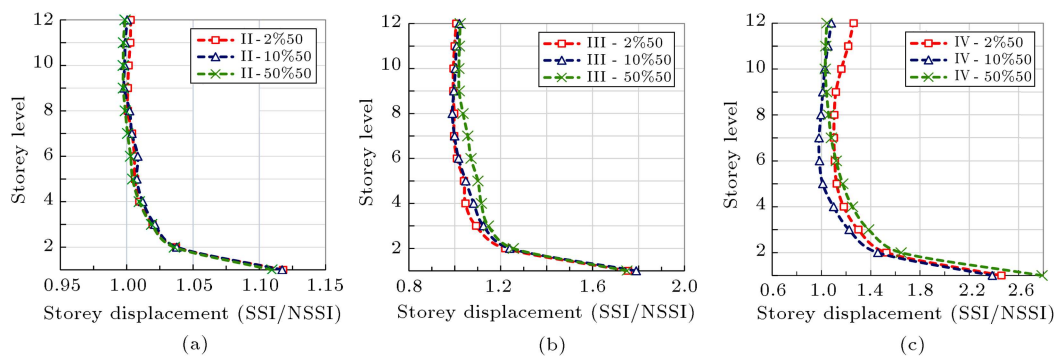


Figure 8. Storey deflection ratio distributions of the 12-storey model for the three hazard levels at site classification: (a) II, (b) III, and (c) IV.

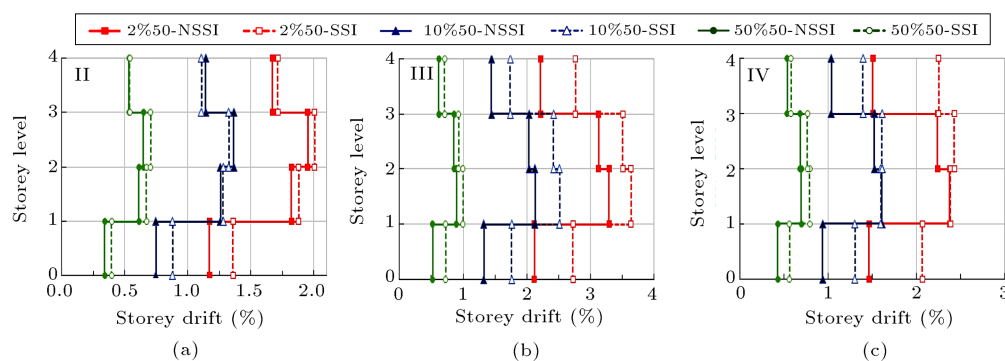


Figure 9. Storey drift ratio distributions of the 4-storey model for the three hazard levels at site classification: (a) II, (b) III, and (c) IV.

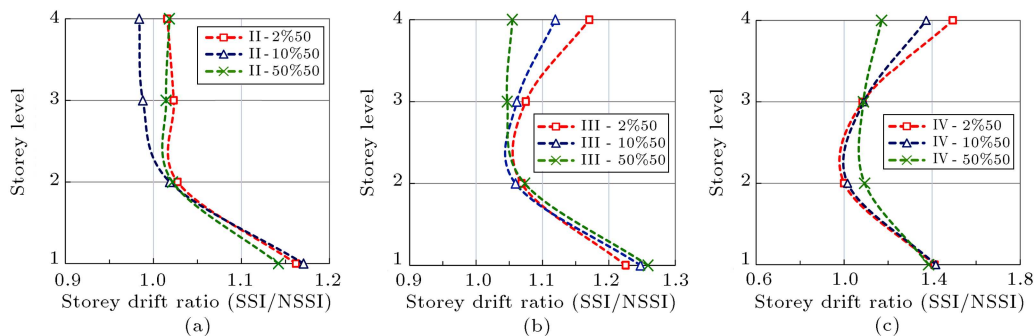


Figure 10. Storey drift ratio distributions to that of fixed-base model in the 4-storey model for the three hazard levels at site classification: (a) II, (b) III, and (c) IV.

describe the state of structures and are classified as Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) for the selected earthquake hazard levels of probability of exceedance of 50%, 10%, and 2% in 50 years, respectively [13]. The above-mentioned three qualitative performance levels are related to the corresponding quantitative maximum storey drift ratios of 0.7%, 2.5%, and 5%, respectively.

The storey drift ratio and the drift ratio of SSI model to that of fixed-based model over the building height are introduced in Figures 9 and 10 for 4-storey building as well as in Figures 11 and 12 for 16-storey building, respectively. The seismic response demands were calculated for different hazard levels and

soil conditions using average envelope of time history analyses. According to Figures 9 and 11, seismic SSI tends to increase the storey drifts in comparison to rigid base conditions. For example, the maximum average drift ratio of the fixed-based 16-storey model for 10% in 50 years ground motions is measured to be 1.1%, 1.8%, and 2.1% for the soil types II, III, and IV, respectively, while the corresponding values for the flexible base are 1.1%, 2.1%, and 2.6%, respectively. In other words, the effects of SSI induce the increases of 0%, 17%, and 24%, respectively, in the simulated storey drifts. Furthermore, the drift amplifications for 2% in 50 years hazard level are more severe than the other levels. As a result, SSI may shift the performance level

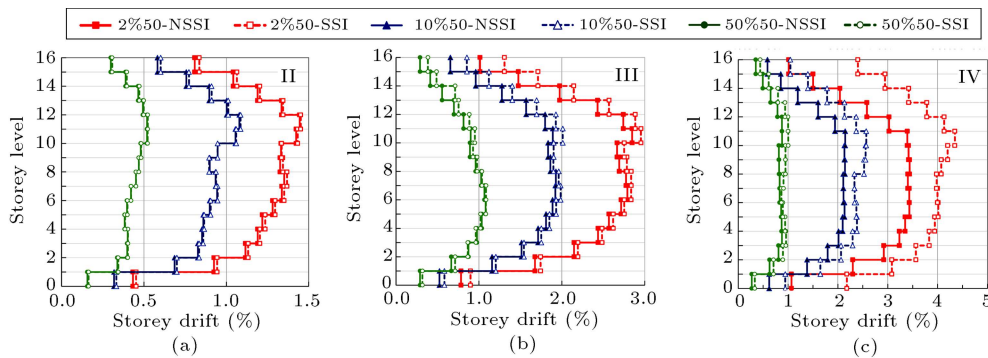


Figure 11. Storey drift ratio distributions of the 16-storey model for the three hazard levels at site classification: (a) II, (b) III, and (c) IV.

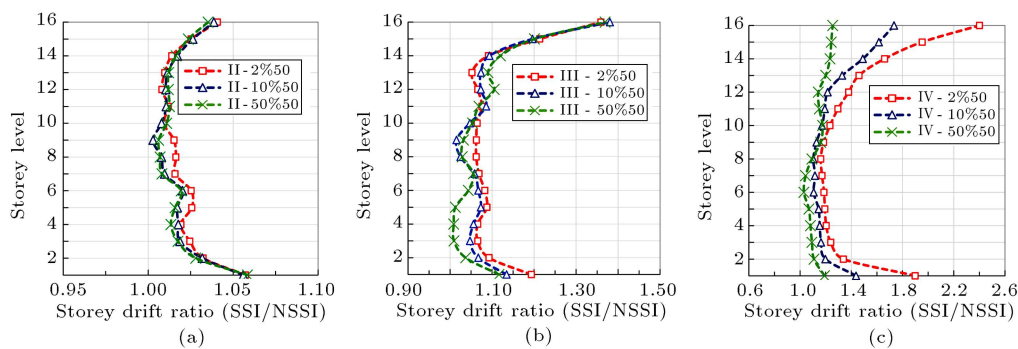


Figure 12. Storey drift ratio distributions of fixed-base model in the 16-storey model for the three hazard levels at site classification: (a) II, (b) III, and (c) IV.

of a structure from life safe zone to near collapse level.

Figure 10(a)-(c) shows the storey drift ratio response distribution over building height compared to that response value of fixed-based model for 4-storey model. The average values of SSI-II, SSI-III, and SSI-IV reach the rate as follows: for 50% in 50 years hazard 1.14, 1.26, and 1.38, respectively; for 10% in 50 years hazard 1.17, 1.25, and 1.4, respectively; for 2% in 50 years hazard 1.16, 1.22, and 1.5, respectively. In a similar way, Figure 12(a)-(c) shows the storey drift response ratio of 16-storey model. The average values of SSI-II, SSI-III, and SSI-IV are: for 50% in 50 years hazard 1.06, 1.37, and 1.29, respectively; for 10% in 50 years hazard 1.06, 1.38, and 1.75, respectively; for 2% in 50 years hazard 1.06, 1.36, and 2.4, respectively. As shown in Figures 10 and 12, the drift demands for flexible strip foundation increase as the underneath soil stiffness decreases. This increasing trend is more significant in the upper and lower storeys. The greatest storey drift ratio's increase occurs for the highest intensity motions (2% in 50 years). Also, the SSI has more significant effect on storey drift ratio as the number of storeys in the model increases. Since the drift demand is an important parameter for the design of structural members, it is very likely that the members are designed conservatively in the absence of incorporation of nonlinear SSI. Such a big difference in storey displacements and drifts is not negligible; thus, the effect

of SSI must be taken into account in dynamic analyses.

It has been reported that the structure supported by deep foundation experiences less storey drifts in comparison to the structure supported by the shallow foundation due to the reduced rocking components [43,44]. They concluded that the choice of foundation type is dominant and should be included while investigating the influence of soil-structure interaction on the performance of mid-rise buildings sitting in soft soil. It is worth mentioning that the complexity of seismic soil-pile-structure interaction problem results in simplifying the presence of pile foundations for the structural design [45, 46]. According to Tahghighi and Konagai [46], a rational method was proposed to define a nonlinear Winkler p-y element which can model the behavior of pile group foundation subjected to lateral loading. Therefore, the proposed simplified p-y approach may conveniently be considered as a further study on the seismic behavior of building frames supported by deep foundations.

5.3.3. Storey shear

The seismic response of structure in terms of storey shear is selected as one of the most important response parameters in seismic design practice. Figures 13 to 16 present the average structural shear demands in comparison to rigid base conditions. The effect of SSI on the inter-storey shear profile over height was taken

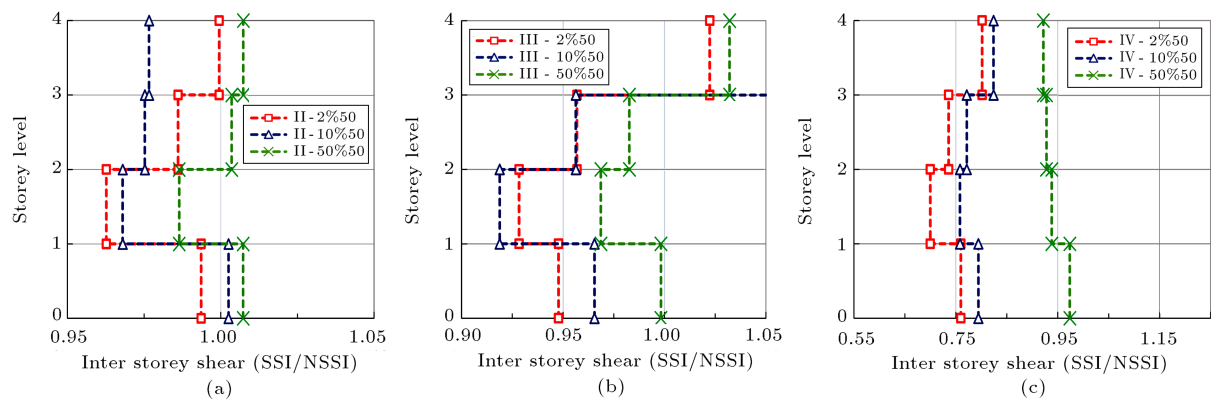


Figure 13. Storey shear ratio distributions of the 4-storey model for the three hazard levels at site classification: (a) II, (b) III, and (c) IV.

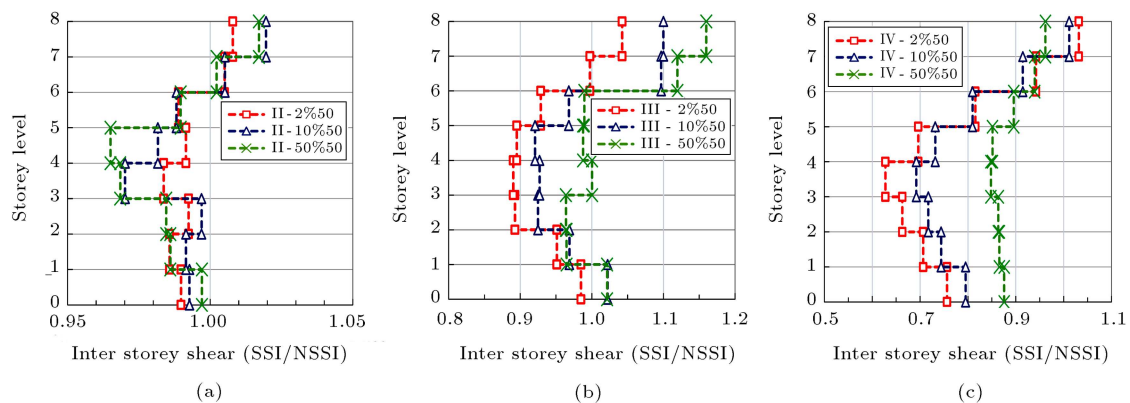


Figure 14. Storey shear ratio distributions of the 8-storey model for the three hazard levels at site classification: (a) II, (b) III, and (c) IV.

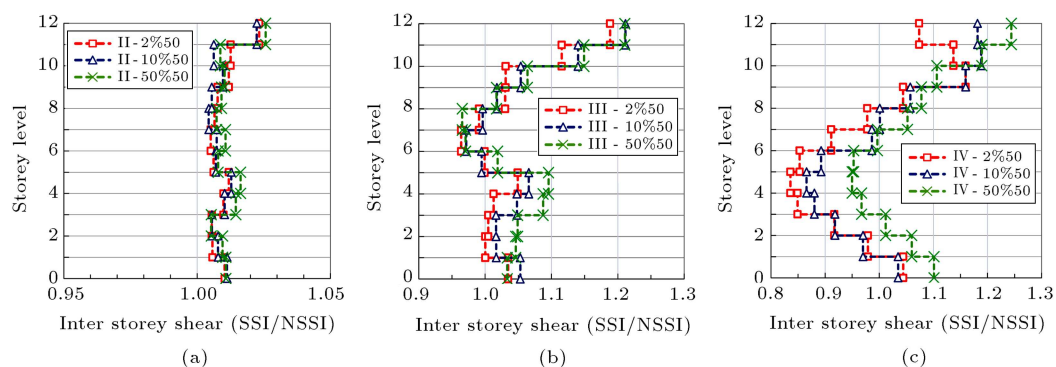


Figure 15. Storey shear ratio distributions of the 12-storey model for the three hazard levels at site classification: (a) II, (b) III, and (c) IV.

into account for all the hazard level ground motions. It could be observed that the effect of higher intensity motions (10% in 50 years and 2% in 50 years) can lead to the potential for greater reduction in structural shear forces due to the capacity mobilization of a larger number of mechanistic springs at the soil-foundation interface.

In Figures 13 and 14, the shear demand ratio of low-rise MRFs (i.e., 4- and 8-storey models) is almost less than one, which has good conformity to the

seismic regulations such as NEHRP [10] and Standard No. 2800 [30], while the shear demands of flexible-base are greater than rigid base conditions for the mid-rise models (i.e., 12- and 16-storey models) (see Figures 15 and 16). In addition, the peak shear at the base of the columns reduces as much as 25% for the low-rise frames, when nonlinearity is considered at the soil-foundation interface. However, for 12- and 16-storey models, the peak base shear increases as much as 17% when soil flexibility is considered. Depending

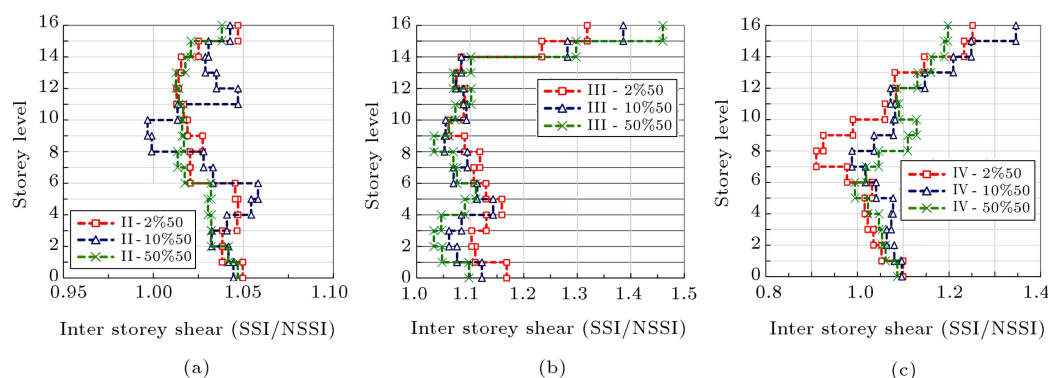


Figure 16. Storey shear ratio distributions of the 16-storey model for the three hazard levels at site classification: (a) II, (b) III, and (c) IV.

on the first result, rigid base assumption would lead to an over-conservative estimation of the base shear, especially as the soil gets softer. Depending on the second result, modeling the soil-foundation interface as fixed can exhibit an under-conservative estimation of the base shear. This implies that the change in base shear is related to the change in the chosen structure and/or ground motion. Thus, the obtained results in this article show that the earthquake response evaluation of a structure can be based on a quite different scenario from those indicated in ordinary seismic codes in which only base shear is reduced for the effect of SSI. Hence, additional work in this area of design guidelines is warranted.

6. Concluding remarks

In this study, the effects of seismic SSI using earthquake motions of different hazard levels were analyzed for typical shallow foundations supporting multi-storey buildings on various soil conditions. Three methods of analysis were used for seismic demands evaluation of the target MRFs. The results of the analyses include fundamental period, storey deflection, storey drift, base shear, and inter-storey shear in flexible-base and fixed-base conditions. A nonlinear Winkler-based approach was adopted for this purpose. The results were compared with those obtained from rigid base models. It was concluded that the dynamic SSI plays a considerable role in seismic behavior of low-to-mid-rise building frames including substantial increase in the lateral deflections, inter-storey drifts, and changing the performance level of the structures. If SSI is not taken into account in the analysis and design properly, the accuracy in assessing the structural safety will not be reliable while facing earthquakes. In other words, the conventional design procedures, excluding SSI, may not be adequate enough to guarantee the structural safety in building frames. Thus, it is essential to consider the SSI effects on the seismic design of regular MRFs, particularly when resting on soft soil deposit. Based on

this work, the following specific conclusions are drawn:

- It is observed that the foundation flexibility has little effect on the fundamental period of MRF systems, and can thus be neglected in evaluating the eigenvalue properties of such systems;
- The obtained findings indicate that performance level of the model resting on soil type II ($375 < V_s < 750$ m/s) does not change substantially, while performance level of the model, resting on soil type III ($175 < V_s < 375$ m/s) and type IV ($V_s < 175$ m/s), substantially increases. Thus, it is essential to consider the SSI effects on the seismic design of steel MRFs resting on soil types III and IV, particularly for high intensity earthquake motions;
- Inter-storey and base shear in the low-rise MRFs almost are reduced when nonlinearity at the soil-foundation interface is considered. But, for the mid-rise buildings, the shear demands are observed to increase when SSI effects are incorporated;

As mentioned, the results obtained in this article will be helpful to quantify the effects of SSI on the structural response, so that designers can be aware of the likely impact of their decisions. However, this study still needs to be verified for additional structures with a wider range of natural periods and relatively larger numbers of appropriate ground motions before the findings could be generalized and used for design recommendations. Furthermore, the choice of pile foundation may be included when investigating the effects of the soil-structure interaction on the performance of mid-rise buildings sitting in high-seismicity soft soil sites.

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