

Increasing Nonlinear Behavior at the Soil-foundation Interface to Improve Seismic Performance of Structures

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Abstract:

It is anticipated that the performance of the structure is influenced by nonlinear behavior at the soil-foundation interface. Results of the previous studies have shown that the vertical Safety Factor (FS) of the foundation is not a suitable criterion to estimate the effectiveness of soil-foundation nonlinearity on the performance of structures. To investigate the subject, three frames were designed and placed on foundations in the current study. The soil was simulated using by distributed Winkler elements considering Q_z behavior. For assessing the ductility demand of beams and columns along with story drift ratios, the soil-structure systems were subjected to artificial records and also some actual ground motion records. The response spectra of the artificial records have been matched with the design spectrum. It was observed that by decreasing FS from 5 to 2, the ductility demands of the elements and drift ratios of the stories decreased significantly in some cases, while the influence was not noticeable in other cases. It was concluded that soil-structure systems can be divided into two parts. By introducing an index, it was shown that if the ratio of the moment capacity of the soil-foundation part to the moment capacity of the super-structure is less than one, the performance of the super-structure can be improved because it does not receive energy higher than the capacity of the soil-foundation part. Whenever the moment ratio is greater than one, the super-structure does not take noticeable advantages from soil nonlinearity, even if low FS is considered for the foundation

1. Introduction

The performance-based design aims to design structures with predictable seismic performance during earthquakes. There are a variety of parameters that can affect the response of structures such as dynamic properties of systems, characteristics of the ground motions, the load-bearing capacity of the systems, etc. One of the main issues that affects both dynamic properties and load-bearing capacity of systems is nonlinear soil-structure interaction which has been paid less attention to.

In this regard, the investigations in the field of seismic performance of structures can be divided into three groups. First, some studies consider the structure to be fixed at the base. For instance, Veletsos et al. [1] and Veletsos and Newmark [2] investigated the response of elastic and

inelastic single-degree-of-freedom systems. Cruz and Chopra [3] have studied the elastic response of building frames. Salazar and Haldar [4] evaluated the codes and standards of Mexico to evaluate the effects of the vertical component of earthquake excitation on the response of steel frames. Chintanapakdee and Chopra [5] assessed the seismic response of vertically irregular frames, which had fixed bases. They concluded that irregularities in the story stiffness and strength could increase the drift of the stories. Ganjavi et al. [6] studied the drift demands of plastic-designed steel frames to evaluate ground motion scaling methods. The aforementioned researches were just a few studies about the seismic performance of fixed-base structures among the others.

The second group of the studies has focused on the effects of elastic soil-structure interaction which dates back to the early 1970s. Jennings and Bielak [7] showed that the response of structures located on elastic half-space would be different in comparison with fixed base structures and the

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soil will increase the fundamental period of the systems. Bielak [8] assessed the dynamic properties of soil-structure systems with embedded foundations. The study indicated that systems would have higher periods and damping as a result of soil-structure interaction effects. Meek and Wolf [9, 10] used a cone model consisting of concentrated springs and dampers to assess the response of an equivalent single degree of freedom which was considered as the structure. Aviles and Perez-Rocha [11] studied the effects of soil-structure interaction of linear structures. They assumed that the behavior of the soil was linear or equivalent linear. Mahsuli and Ghannad [12] investigated inelastic structures located on embedded foundations using the cone model. Khoshnoudian et al. [13] suggested inelastic displacement ratios for soil-structure systems. Chougule and Dyavanal [14] considered rigid and flexible foundations to assess the effects of soil-structure interaction. The soil was simulated by a set of elastic springs. They showed that the flexibility of the soil could reduce base shear demands. Gholamrezatabar et al. [15] estimated inelastic displacement factors for multi-degree of freedom systems considering soil-structure interaction and higher mode effects. They concluded that incorporating higher mode effects reduces the ductility demand of the systems. Ayough and Taghia [16] investigated the response of steel moment and braced frames laid over soil subjected to near-source ground motions.

In the third group of studies, nonlinear behavior has been assigned to the soil. Halabian and El Naggar [17] investigated the effects of nonlinear soil-structure interaction on the seismic response of tall structures using the Finite Element Method (FEM). They concluded that the secondary soil nonlinearity depends on many parameters like the type of structure, the frequency content of excitation and the dynamic properties of the soil. Raychowdhury [18] checked the effects of soil parameters uncertainty on the seismic demands of steel frames. Anastasopoulos et al. [19] proposed that inelastic behavior and failure in the soil can be used to isolate the structure from the energy of base excitation. Raychowdhury [20] analyzed several steel moment frames under base excitations, incorporating the nonlinear behavior of soil. Gelagoti et al. [21] suggested the idea of rocking isolation by considering the nonlinear behavior of soil for low-rise buildings on single foundations. Deng et al. [22] conducted probabilistic analyses to find the effects of rocking isolation on columns of bridges. Masaeli et al. [23] checked the idea of rocking isolation for tall buildings subjected to near-fault ground motions considering bidirectional excitations. Ghannad and Jafarih [24] developed equations for inelastic displacement ratios for nonlinear soil-structure systems allowed to uplift. Zubair and Shilpa [25] conducted parametric analyses to determine the effect of nonlinear soil-structure interaction of raft

foundations. Abdollahiparsa et al. [26] studied the effects of the vertical component of excitations on steel frames considering soil-structure interaction. Homaei et al. [27] assessed the performance of steel buildings. They concluded that the flexibility of the foundation and irregularity in the height of the frames could increase the seismic demand for lower stories. Mathew et al. [28] concluded that soil-structure interaction could reduce seismic demands of reinforced concrete frames up to 50 percent for story drift. Star et al. [29] found that the nonlinear behavior of the soil would increase the effective period of systems under forced vibration tests. Jafarih and Ghannad [30, 31] concluded that ductility demand in the single-degree-of-freedom systems could be reduced by allowing soil yielding and foundation uplifting. Akhoondi and Behnamfar [32] drew seismic fragility curves for steel buildings considering nonlinear soil-structure interaction. Vaseghiamiri and Ghannad [33] investigated the importance of super-structure modeling in estimating soil contribution to the seismic response of soil-structure systems. Sadjadi et al. [34] conducted a parametric study to explore the key parameters affecting the seismic performance of deformable rocking soil-structure systems subjected to pulse-type excitations. Jafarih et al. [35] presented a modified simple soil model to use in the fish-bone model.

As mentioned above, many studies investigated the performance of fixed-base structures and soil-structure systems. There are indeed several pieces of research that incorporate the nonlinearity of soil behavior in the analyses. For elastic soil-structure systems or equivalent elastic soil-structure systems, the influence of soil structure interaction can be well described through non-dimensional parameters like period ratio and aspect ratio [24]. For nonlinear soil-structure systems, only a vertical safety factor is suggested to control the level of nonlinearity at the soil-foundation interface. Previous studies [30, 31] have shown that decreasing the safety factor has not the same influence on the performance of different structures. So, it is needed to propose a new parameter by which the variation of the seismic demands of structures as a result of the soil-foundation nonlinearity could be described by it.

In the current study, several frames were designed and placed over various soil-foundation systems considering different safety factors and shear wave velocities. Then the soil-structure systems were subjected to different base excitations to evaluate the seismic performance of the structures considering nonlinear behavior at the soil-foundation interface. Finally, a new parameter has been proposed, and the variations of the seismic performance of the super-structure due to nonlinear behavior at the soil-foundation interface could be described by it.

2. Model Buildings

In this study, three frames were designed and placed on different foundation types while nonlinear behavior is considered at the soil-foundation interface. The details of the soil-structure models are mentioned in the following sections.

2.1 Design of steel frames

As shown in Figure 1, three model buildings with steel moment-resisting frames are considered to be assessed. Each of them has 5, 8, and 11 stories, respectively. For designing the model buildings, seismic loads were determined based

on ASCE-7 [36] considering special moment frames ($S_1 = 1, S_{DS} = 1.6, R = 8$ and $I_e = 1$). Then, they were designed based on AISC-341 [37]. Afterwards, an interior frame of each building was selected and modeled in the open-source software OpenSees [38] (Open System for Earthquake Engineering Simulation) of the Pacific Earthquake Engineering Research Center to evaluate its seismic performance under various conditions subjected to excitation. To model the sections of the frames, fiber sections are considered for modeling beams and columns, and steel material with bilinear behavior was assigned to them. The fundamental period of selected frames for 5-, 8- and 11-story are 0.8, 1.2, and 1.8 sec, respectively.

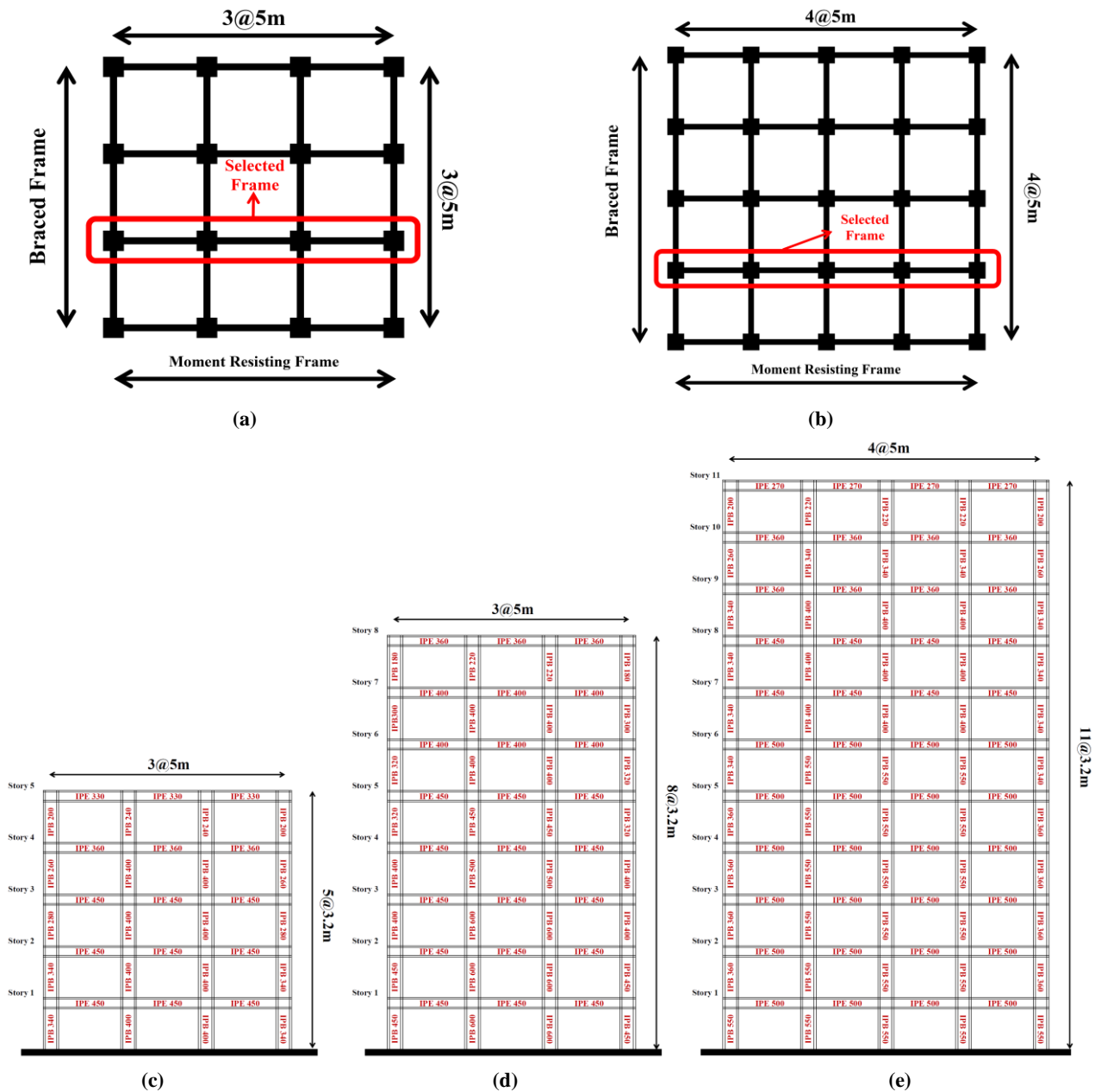


Fig. 1: Details of the frames of the model buildings: (a) Plan of 5- and 8-story models; (b) Plan of 11-story model; (c) Selected frame of the 5-story model; (d) Selected frame of the 8-story model; (e) Selected frame of the 11-story model.

2.2 Soil-structure model

In this study, the frames are constructed over strip foundations which are laid on the Winkler foundation. As shown schematically in Figure 2, vertical stiffness and damping of the soil are modeled by using distributed zero-length elements which include a combination of springs and dampers called QzSimple [39]. The behavior of the QzSimple material is shown in Figure 2(b). It can be seen that QzSimple material has the capability of modeling soil yielding and uplifting.

The model parameters of the Winkler foundation, including the behavior of Qzsimple material, were calibrated by Harden et al. [39] through centrifuge tests considering a broad range of design vertical safety factors for the models and by considering both clay and sand soil mediums. Numerical results showed a reasonable agreement between the nonlinear Winkler-based approach and the experimental data for moment, settlement, and lateral displacement. A concentrated spring and damper are connected to the center of the foundation to model sway stiffness and damping of the foundation.

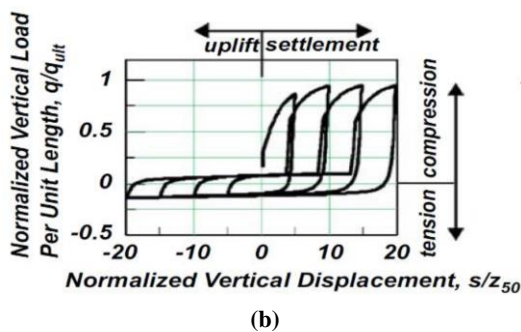
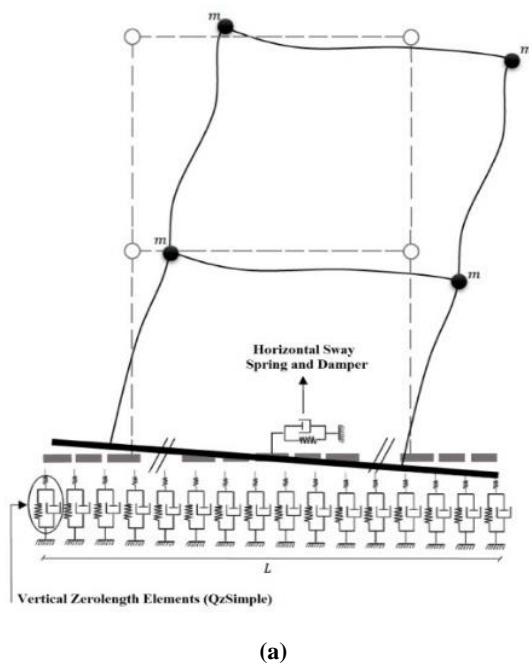


Fig. 2: Schematic configuration of soil-structure model: (a) Soil-structure model; (b) QzSimple material behavior [39].

Based on the FEMA-440 [40], the coefficients of spring (K_x) and damper (C_x) of Sway direction are estimated as follows:

$$K_x = \frac{8Gr_x}{2 - \vartheta} \quad (1)$$

$$C_x = 0.65 \left(\frac{K_x r_x}{V_s} \right) \quad (2)$$

Where G is the shear modulus of the soil, V_s is the shear wave velocity of the soil, r_x is the equivalent radius of the foundation for sway motion equals to $\sqrt{A_f/\pi}$ in which A_f is the area of foundation and ϑ is Poisson's ratio of the soil.

According to the equations presented in FEMA-440 [40], the rocking stiffness and damping of the foundation can be determined as below:

$$K_\theta = \frac{8Gr_\theta^3}{3(1 - \vartheta)} \quad (3)$$

$$C_\theta = 0.4 \left(\frac{K_\theta r_\theta}{V_s} \right) \quad (4)$$

where r_θ is the equivalent radius of the foundation for rocking motion equals to $\sqrt[4]{4I_f/\pi}$ in which I_f is the moment of inertia of the foundation about the corresponding rocking axis. To find the coefficients of vertical springs and dampers, ASCE-41 [41] has considered the soil-structure systems with concentrated rocking and sway springs and dampers as the benchmark. It has allowed the distribution of the stiffness and damping under the foundation in a reasonable way that leads to the same dynamic properties as the benchmark model. So, in this study, the rocking stiffness and damping are distributed uniformly as follows in Equations 5 and 6. Results of the studies have shown that the period and the response of the soil-structure systems under base excitation are the same for both soil-structure systems with distributed soil modeling and concentrated soil modeling [30,31]. The intensity of stiffness and damping of vertical springs and dampers (per unit area) can be estimated using Equations 5 and 6 as follows:

$$K_v = \frac{K_\theta}{I_f} \quad (5)$$

$$C_v = \frac{C_\theta}{I_f} \quad (6)$$

The ultimate capacity of the soil (q_{ult}) is estimated following the vertical safety factor as mentioned in Equation 7.

$$q_{ult} = \frac{W}{A_f} FS \quad (7)$$

Where W is the seismic weight of the soil-structure system and FS is the safety factor.

It should be noted that the soil-foundation model which is utilized in the current study is the same as the model used in the studies of Jafarih and Ghannad [30, 31]. Also, the moment-rotation curve of the nonlinear soil-foundation model in the current study shows good agreement with the results of the formulations presented by Allotey and El Naggar [42] for a sample of soil-foundation which is shown in Figure 3.

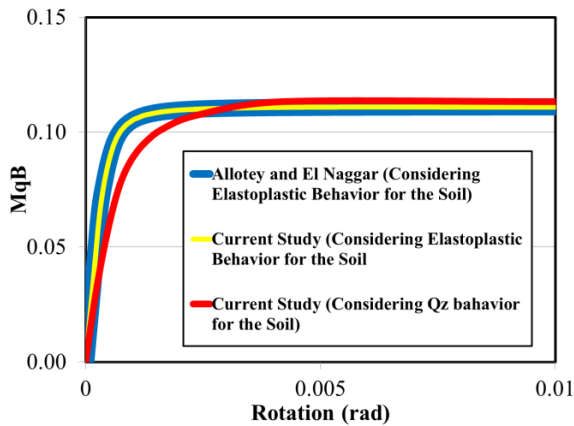


Fig. 3: Comparison of the moment-rotation curve of the nonlinear soil-foundation model in the current study and results of the formulations presented by Allotey and El Naggar [42] for a sample of soil-foundation

3. Methodology

As mentioned, this study aims to evaluate the effects of the soil-foundation nonlinearity on the performance of the super-structure under different conditions considered for the foundation. To reach this goal, two shear wave velocities ($V_s=100$ m/s and 365 m/s) and three safety factors ($FS=2, 3$ and 5) are assigned to the foundations. Then the responses of the super-structure as a part of the soil-structure system are compared with corresponding fixed base frames.

Two main parameters which have a considerable effect on the response of soil-structure systems are aspect ratio (H/b) which is the ratio of the height of the structure (H) to the half of the foundation length (b) and period ratio (T_{ssi}/T_{fix}) which is the ratio of the soil-structure system period (T_{ssi}) to the period of the structure when it is considered fixed at the base (T_{fix}).

It is well known that the effect of soil-structure interaction will increase by increasing the period ratio of soil-structure systems (T_{ssi}/T_{str}) [41]. For any specific system, this can be done by decreasing the soil stiffness beneath the foundation. In this study, it is assumed that for any real soil-structure system, the soil beneath the foundation is unchangeable. So, for each soil-structure system laid over the soil with specific shear wave velocity, the dimensions of

the foundations are reduced (for example the width of the strip foundation can be reduced) to decrease the vertical safety factor of the foundation.

According to Equations 1 and 3, a reduction of the foundation dimensions will lead to a decrement in the foundation stiffness which consequently increases the period ratio (T_{ssi}/T_{str}). In other words, the reduction of foundation dimensions can affect the performance of the super-structure through two aspects. First, the nonlinear behavior at the soil-foundation interface will increase, and second, the effect of soil-structure interaction increases because of the enhancement of the period ratio. The variation of T_{ssi}/T_{str} by changing FS and V_s is shown in Table 1 for different soil-structure systems. In this study, the density of the soil is considered to be $\rho = 1700 \text{ kg/m}^3$ and also, it is assumed that the Poisson's ratio equals $\nu = 0.4$ for the soil. Also, the backbone curve of the clay soil is selected for Qzsimple material

ASCE-7 [36] considers the spectra corresponding to hazard levels of 2% and 10% probability of occurrence in 50 years as MCE and DBE spectra respectively. In ASCE-7 [36] the DBE spectrum is drawn for earthquake demands that are 2/3 of MCE ground motions.

For conducting time history analyses, two collections of ground motions are selected. The first collection includes three artificial records whose response spectra match the DBE spectrum and can be scaled to match the MCE spectrum, as well. The artificial records were obtained using SeismoMatch software [43]. Also, a collection of 12 real ground motions which were registered on site class C, was selected. The real ground motions were scaled using the ASCE-7 [36] procedure due to the DBE spectrum. The response spectra of the three artificial records and 12 real ground motions are shown in Figure 4. The characteristics of 12 real ground motions are presented in Table 2.

4. Assessment of the seismic performance of building models

As mentioned, the performance of the building models is evaluated under different conditions while they are subjected to ground motions. At first, it is assumed that the systems are fixed at the base and then the changes in the responses of systems are studied while different soil-foundation conditions are considered.

4.1 Seismic performance of fixed base frames under artificial records

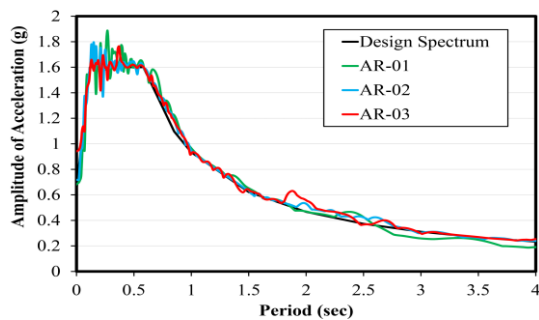
The frames are subjected to the three artificial records considering two hazard levels (DBE and MCE). Then, the ductility demands of the beams and columns are calculated. It is clear that, in moment frames, the plastic hinges are formed at the ends of beams and columns under base excitation.

Table 1: Period ratio and aspect ratio of soil-structure system.

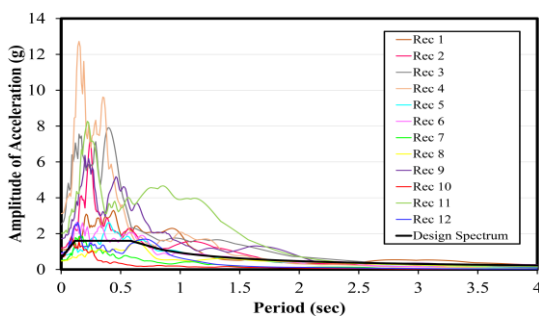
		5-story model			8-story model			11-story model		
T_{fix}		0.830			1.196			1.791		
Aspect ratio (H/b)		2.098			3.357			3.477		
FS		2	3	5	2	3	5	2	3	5
$V_S=365m/s$	T_{ssi}	0.840	0.937	0.834	1.221	1.212	1.204	1.832	1.817	1.804
	T_{ssi}/T_{fix}	1.012	1.129	1.005	1.021	1.013	1.007	1.023	1.015	1.007
$V_S=100m/s$	T_{ssi}	0.957	0.911	0.870	1.517	1.409	1.311	2.148	2.127	1.971
	T_{ssi}/T_{fix}	1.153	1.098	1.048	1.268	1.178	1.096	1.199	1.188	1.101

Table 2: Selected real ground motions' characteristics [40]

Record Number	Date	Earthquake Name	Magnitude (Ms)	Station Name	Station Number	Component (deg)	PGA (cm/s ²)
01	10/17/1989	Loma	7.1	APEEL 7, Pulgas	58378	0	153
02	10/17/1989	Loma	7.1	Gilroy #6, San Ysidro Microwave site	57383	90	166.9
03	10/17/1989	Loma	7.1	Gilroy, Gavilon College Phys Sch Bldg	47006	67	349.1
04	10/17/1989	Loma	7.1	Santa Cruz, University of California	58135	360	43.1
05	10/17/1989	Loma	7.1	San Francisco, Diamond Heights	58130	90	110.8
06	10/17/1989	Loma	7.1	Fremont, Mission San Jose	57064	0	121.6
07	10/17/1989	Loma	7.1	Monterey, City Hall	47377	0	71.6
08	10/17/1989	Loma	7.1	Yerba Buena Island	58163	90	66.7
09	10/17/1989	Loma	7.1	Anderson Dam, Downstream	1652	270	239.4
10	4/24/1984	Morgan	6.1	Gilroy, Gavilon College Phys Sci Bldg	47006	67	95
11	4/24/1984	Morgan	6.1	Gilroy #6, San Ysidro Microwave Site	57383	90	280.4
12	7/8/1986	Palmspring	6	Fun Valley	5069	45	129



(a)



(b)

Fig. 4: Response spectra of artificial and real ground motions: (a) Acceleration response spectra of artificial records and DBE spectrum; (b) Scaled acceleration response spectra of 12 real ground motions and design spectrum.

The ductility demand of each end of the element is calculated as follows:

$$Ductility = \frac{\theta_{max}}{\theta_y} \tag{8}$$

Where θ_{max} is the maximum rotation of the hinge during excitation and θ_y is the yield rotation of the corresponding section. Then, the averages of the ductility demands of beams and columns for each story are estimated. In Figure 5, it can be seen that the average ductility values for beams are much higher than their corresponding values for the columns at each story. Generally, the ductility values for lower stories are greater than upper stories. This is true for all stories, except for the roof story whose its capacity is considerably smaller than lower stories. The ductility pattern over the height of structures is the same as the results of Jafarih and Yekrangnia [44] for models designed based on strength distribution proposed by codes. Moreover, it can be seen that the ductility demands at lower stories increase by increasing the number of stories.

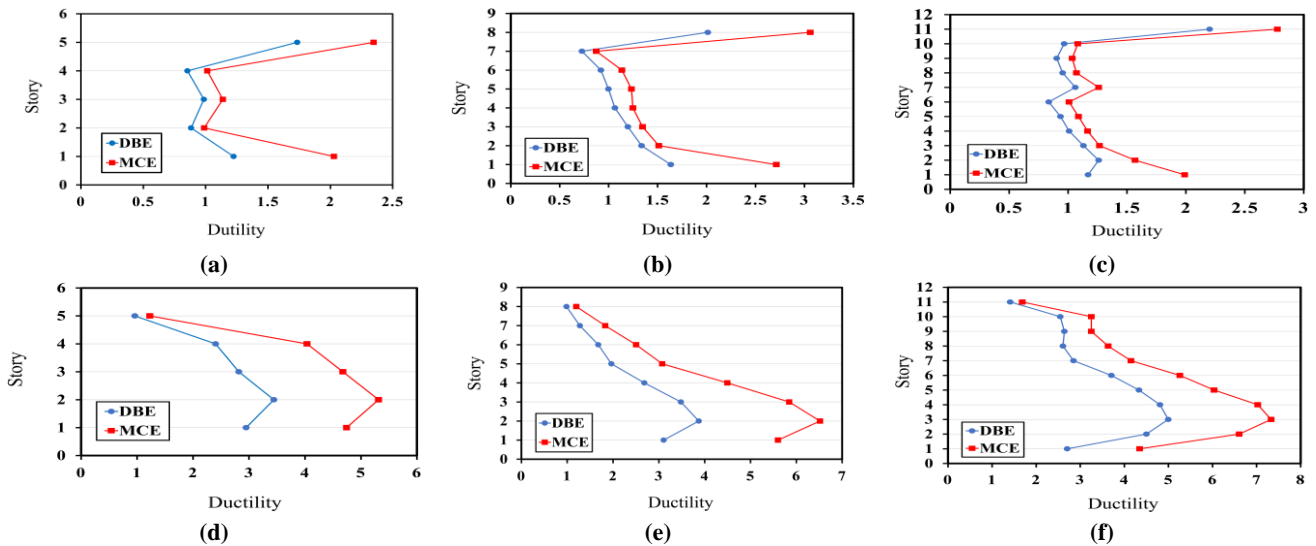


Fig. 5: Average ductility demands of beams and columns of the fixed base frames subjected to three artificial records considering two hazard levels (DBE and MCE): (a) Columns of the 5-story model; (b) Columns of the 8-story model; (c) Columns of the 11-story model; (d) Beams of the 5-story model; (e) Beams of the 8-story model; (f) Beams of the 11-story model.

4.2 Seismic performance evaluation of frames considering nonlinear soil-structure interaction

In this section, frames are laid over a strip foundation. In the first step, a large value for the safety factor is considered against gravity loading for designing the foundation ($FS = 5$). As it was expected, the soil-foundation system experiences very low nonlinear behavior while subjected to artificial records because of the large value of FS . Figure 6 shows that the effect of soil-structure interaction on the ductility demands of stories is not noticeable. It is true not only for soil-structure systems with $V_s = 365$ m/s but also the situation is the same for systems with $V_s = 100$ m/s. According to previous studies [24] the performance of soil-structure systems can be described by studying non-dimensional parameters.

Two main parameters that have a considerable effect on the response of soil-structure systems are aspect ratio (h/b) and period ratio (T_{ssi}/T_{fix}).

It can be shown that by decreasing h/b and increasing T_{ssi}/T_{fix} , the influence of soil on the performance of the structure will increase. It is clear that T_{ssi}/T_{fix} will increase by reducing V_s from 365 m/s to 100 m/s. Calculations showed that for models of the current study, the variation of T_{ssi}/T_{fix} is not considerable because of the large fixed period and large stiffness of the soil-foundation part. Consequently, the effects of elastic soil-structure interaction on the three models which are assessed can be ignored.

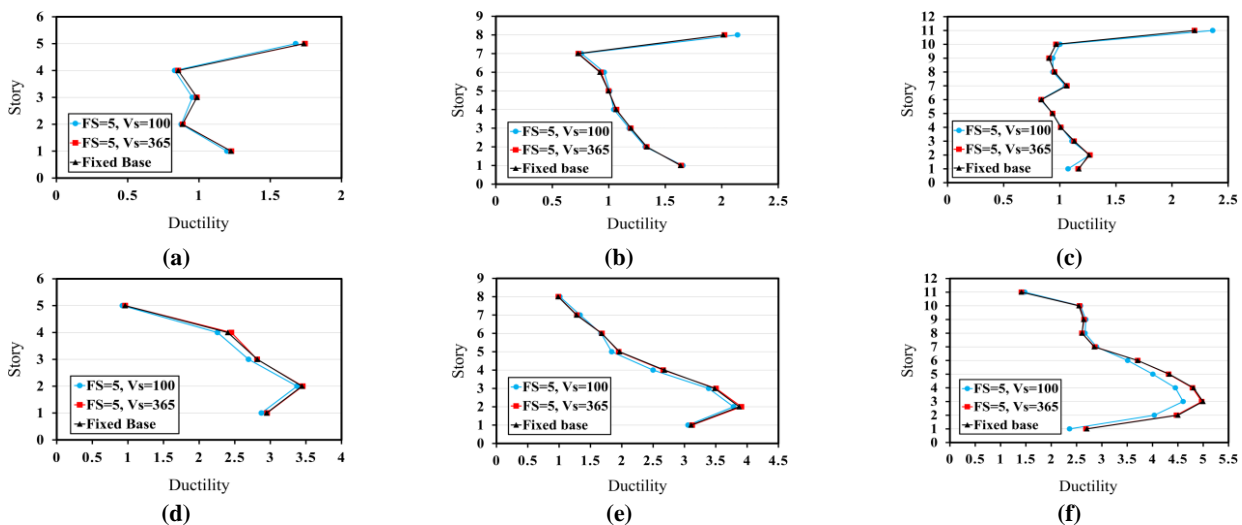


Fig. 6: Average ductility demands of beams and columns of frames laid on strip foundation subjected to three artificial records considering DBE hazard level and $FS=5$: (a) Columns of the 5-story model; (b) Columns of the 8-story model; (c) Columns of the 11-story model; (d) Beams of the 5-story model; (e) Beams of the 8-story model; (f) Beams of the 11-story model.

In the second step, the safety factor for designing the foundation is decreased. As shown in Figure 7, it can be seen that the commonly used safety factor equal to 3 has no considerable effect on the performance of the structure in comparison with the fixed base frames. But lower values of safety factors (for instance $FS = 2$) will reduce the ductility demands of the beams and columns. In other words, by decreasing the safety factor of the foundation and as a result, increasing nonlinear behavior in the soil beneath the foundation, the input energy which is conducted to the super-structure will be dissipated and reduced. So, the performance of the structure can be improved by a reduction in ductility demands.

The influence of the FS is more noticeable for lower stories in comparison with upper stories. Figure 8 indicates that, except for the upper stories, the drift of the middle and lower stories decreases with increasing the nonlinear behavior in the soil beneath the foundation. The percentage of the reduction increases by increasing the intensity of the excitation from DBE to MCE.

Figure 9 shows the permanent settlement under the foundation while soil-structure systems are subjected to artificial records. It should be noted that FEMA-356 [45] has limited the total settlement of the foundations to 6 inches (15.24 cm) for Life Safety. Moreover, it has accepted major settlements and tilting for Collapse Prevention performance level.

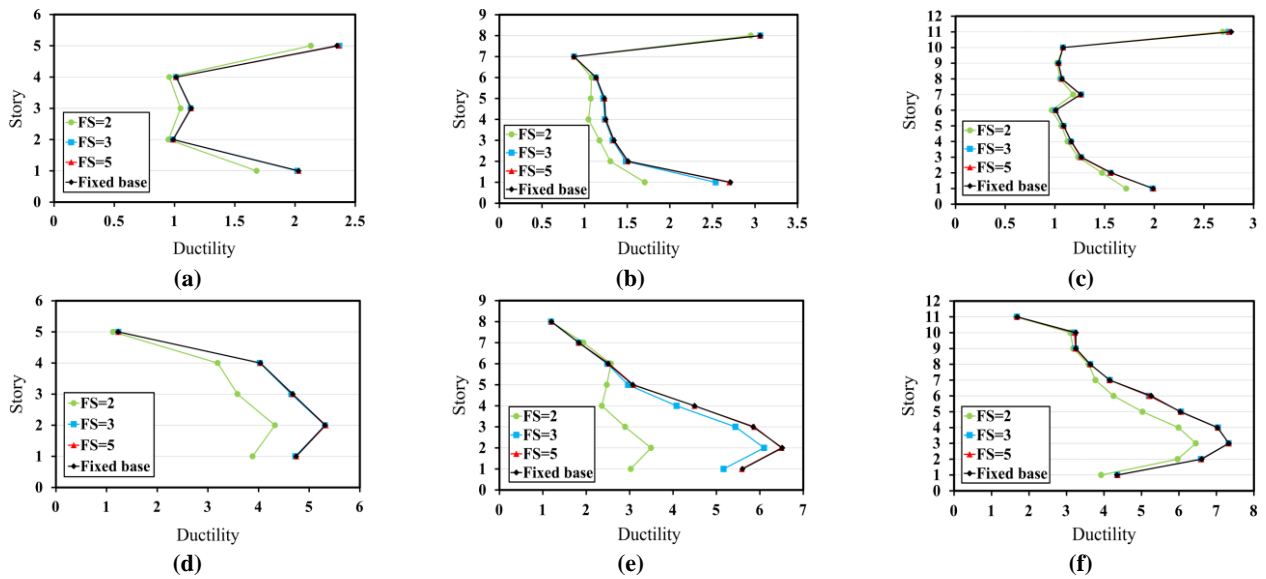


Fig. 7: Average ductility demands of beams and columns of fixed base frames and frames laid on strip foundation subjected to three artificial records considering MCE hazard level and $V_s=365\text{m/s}$: (a) Columns of the 5-story model; (b) Columns of the 8-story model; (c) Columns of the 11-story model; (d) Beams of the 5-story model; (e) Beams of the 8-story model; (f) Beams of the 11-story model.

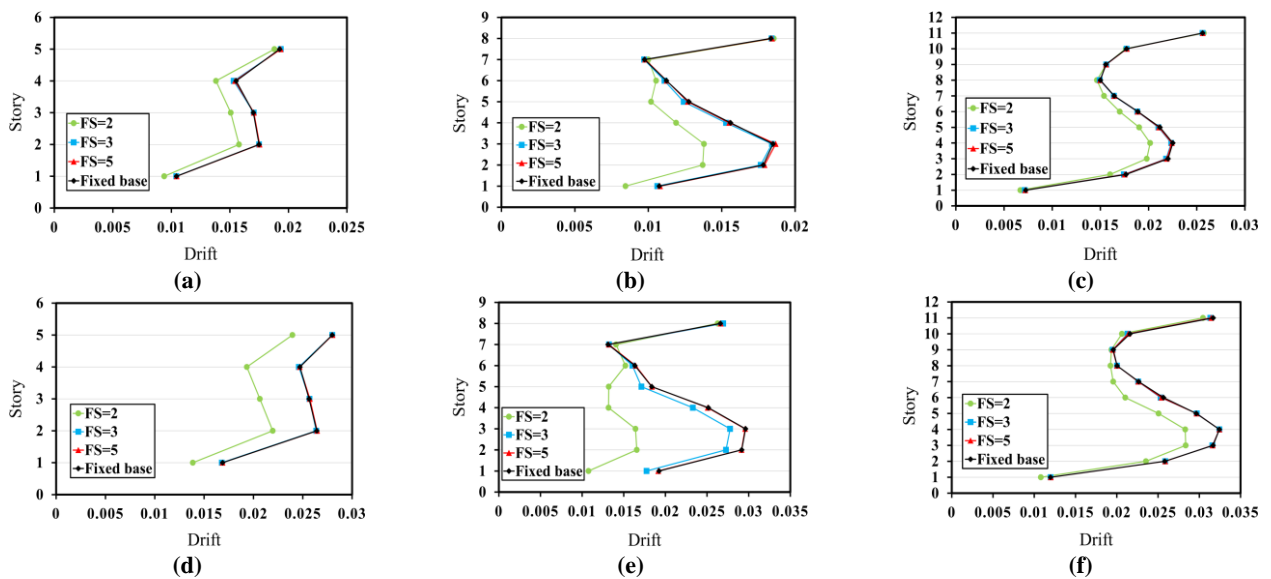


Fig. 8: Average of story drift of fixed base frames and frames laid on strip foundation subjected to three artificial records considering DBE and MCE hazard level and $V_s=365\text{m/s}$: (a) the 5-story model (DBE); (b) the 8-story model (DBE); (c) the 11-story model (DBE); (d) the 5-story model (MCE); (e) the 8-story model (MCE); (f) the 11-story model (MCE).

According to Figure 9, it can be seen that under the DBE hazard level, the settlements of the foundations are smaller than 15.24 cm (threshold of LS).

In Figure 9 it can be seen that the values of the permanent settlement under the foundations for FS=2, are significantly higher than the corresponding values for FS=3 and 5 in all cases. This can be attributed to the Qz behavior model as shown in Figure 2(b). This is because for vertical stresses higher than $0.5q_{ult}$, the tangent on the Qz behavior curve (in other words the effective stiffness of the soil) decreases significantly and Qz behavior becomes significantly nonlinear. This situation takes place for FS=2 while for FS=3 and 5, the stresses on the soil remain smaller than $0.5q_{ult}$, during most of the seismic excitations time.

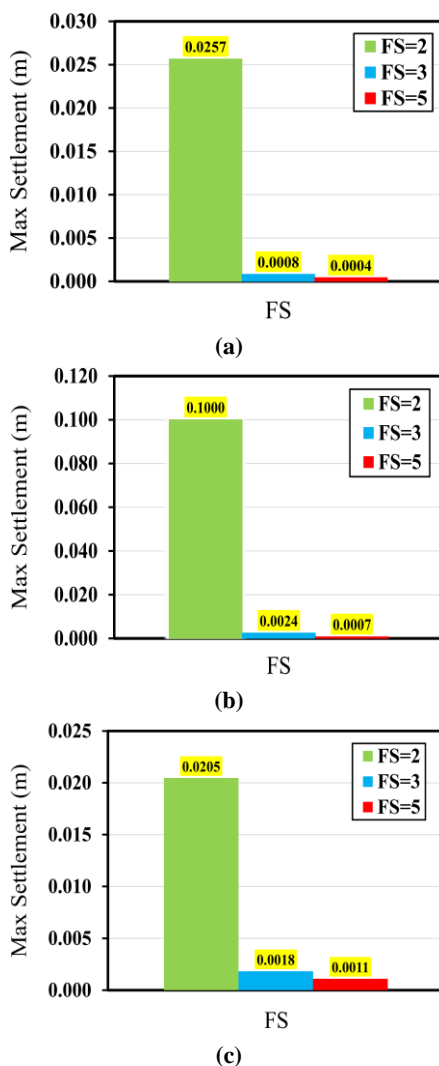


Fig. 9: Average of permanent settlement under the foundation of frames laid on strip foundation subjected to three artificial records considering DBE and MCE hazard level and $V_s=365\text{m/s}$: (a) 5-story model (DBE); (b) 8-story model (DBE); (c) 11-story model (DBE).

In Figure 10, the ratios of the ductility demands (Ductility of beams and columns) are calculated while the shear wave

velocity of the soil is considered 100m/s to their corresponding values when the shear wave velocity of the soil is assumed 365m/s. As it was expected, the period of soil-structure systems increased as a result of the reduction in the soil stiffness and consequently seismic demands decreased. It can be seen that increasing the FS by increment of the foundation dimensions, reduces the influence of changing shear wave velocity from 365m/s to 100m/s which leads the ductility ratios to converge to unity.

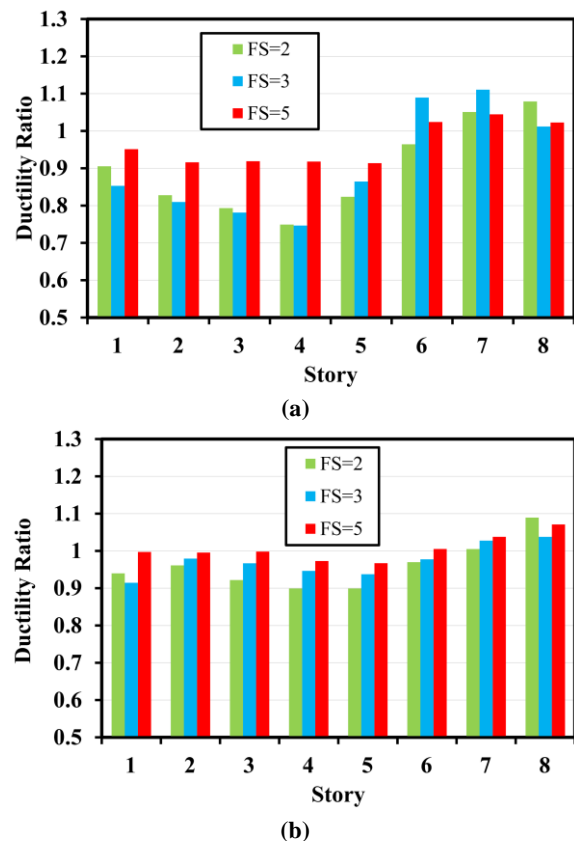


Fig. 10: Seismic demands ratios of 8 story frame laid on strip foundation subjected to artificial records considering DBE hazard level while V_s is considered to be 100m/s to their corresponding values when V_s is assumed to be 365m/s: (a) Ductility ratios of beams; (b) Ductility ratios of columns

The performance of the 8-story frame laid over the strip foundation is evaluated under real ground motions in Figure 11. It can be seen that similar to the results of artificial records, the ductility of beams and columns, and the drift of stories decreases by decreasing the FS. In both collections of the ground motions, the trends in the reduction of the responses are the same in the upper and lower stories. In other words, the influence of the FS is more noticeable for lower stories than for upper stories. Also Figure 11(d) indicates that the variations of the maximum settlements of the foundation (by changing the FS) are the same under both artificial and real ground motions. For both collections of ground motions, the maximum settlements are smaller than the limitation suggested by FEMA-356.

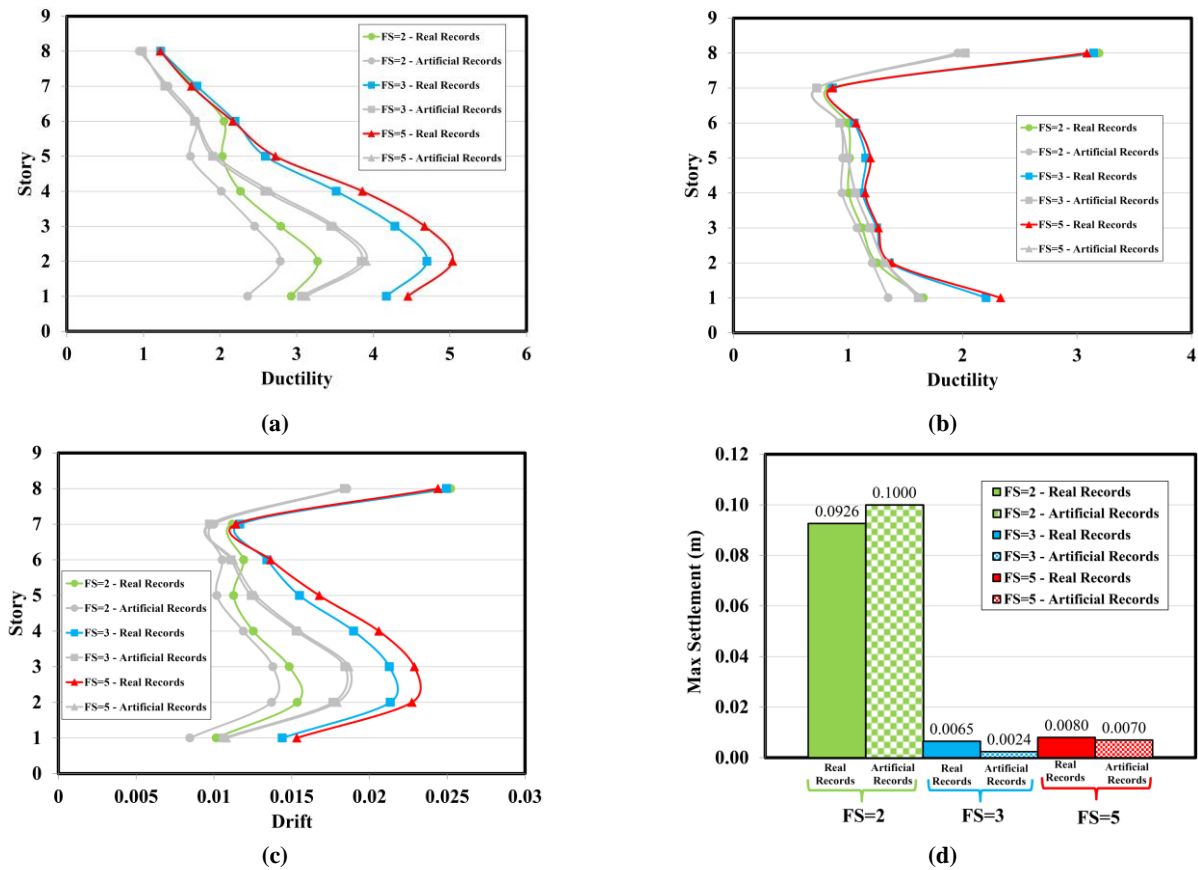


Fig. 11: Comparison of the responses of 8 story frame laid on a strip foundation subjected to real and artificial records considering DBE hazard level and $V_s=365\text{m/s}$: (a) Average ductility demands of beams; (b) Average ductility demands of columns (c) Average of story drift (d) Average of permanent settlement under the foundation.

4. 3 Identification of the main effective parameter

In this section, it is investigated to identify parameters that have more influence on the performance of nonlinear soil-structure systems. It is important to find out the variations of which parameter can improve the performance of the super-structure as a part of the soil-structure system. In Figure 12, the ductility ratios of beams and columns of soil-structure systems with FS=2 are presented. The ductility ratio is defined as the ratio of the ductility of soil-structure elements with FS=2 to its corresponding values in fixed base frames. It can be seen that the effect of nonlinear soil behavior can increase by increasing the intensity of the excitation from DBE to MCE. In other words, soil nonlinearity can reduce the ductility of the elements more by increasing the input energy of earthquakes. This is because the capacity of soil for transmitting input energy to the super-structure is limited. Also, the influence of nonlinear behavior in the soil is more considerable for the beams in comparison with the columns. Moreover, soil nonlinear behavior decreases the ductility demands of the elements of lower stories more in comparison with upper stories. Since the path of input energy goes through the base to the top stories, upper stories do not receive more energy than the capacity of lower stories.

As shown in Figure 12, the influence of the soil is not the same for all frames and its effect is more noticeable for the 8-story model in comparison with two other frames. Also, for all building models in the current study, the influence of soil nonlinear behavior becomes much more noticeable by reducing FS from 3 to 2 in comparison with the situation where the FS is reduced from 5 to 3. In other words, the performance of the structure does not change gradually by reducing FS from 5 to 2. So, it can be concluded that FS is not a suitable parameter for forecasting and expressing the level of the soil nonlinear behavior effect. Thus, it seems that there is a need to introduce a new index, in which the influence of the soil nonlinear behavior on the super-structure could be described by it. According to Table 1, although the aspect ratio is the lowest for the 5-story model which can lead to higher radiation damping for this model in comparison with the other models, the value of the period ratio (T_{ssi}/T_{fix}) is near unity for all frames located on soil with $V_s = 365 \text{ m/s}$. So, it can be concluded that the effect of **elastic** soil-structure interaction can be neglected for all three frames in this study. Therefore, the non-linear behavior at the soil-foundation interface has a prominent role that affects the performance of the structure.

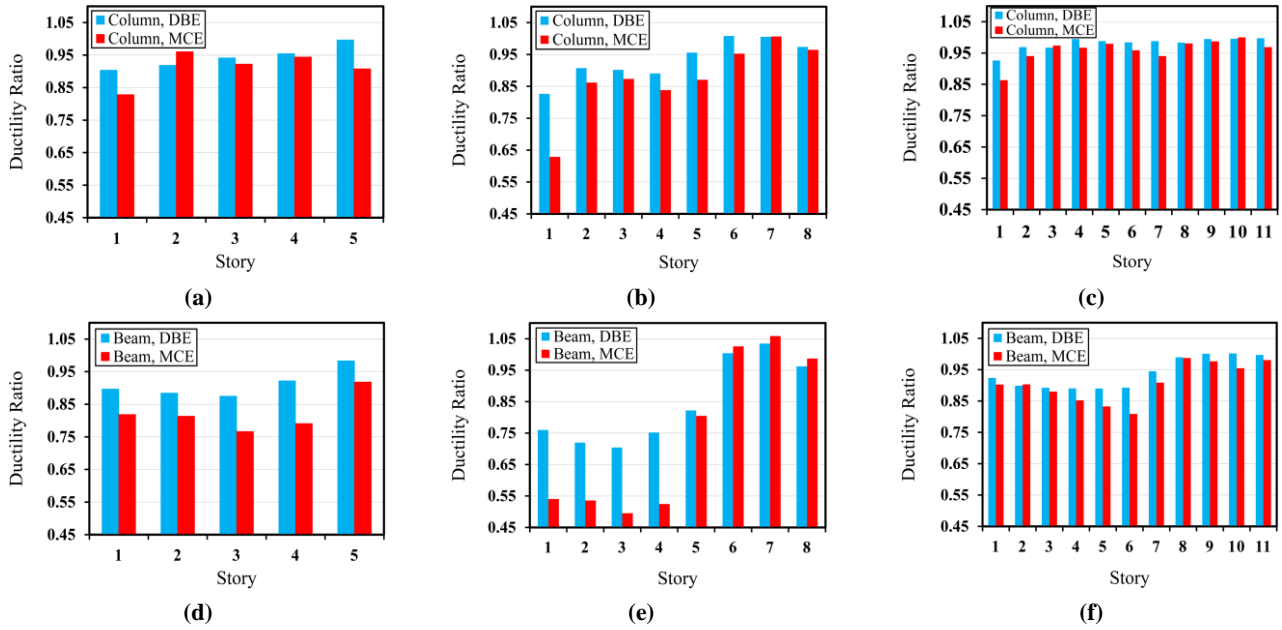


Fig. 12: Ductility ratios of beams and columns of soil-structure systems with FS=2 to their corresponding values in fixed base frames subjected to three artificial records considering two hazard levels and $V_s=365\text{m/s}$: (a) Columns of the 5-story model; (b) Columns of the 8-story model; (c) Columns of the 11-story model; (d) Beams of the 5-story model; (e) Beams of the 8-story model; (f) Beams of the 11-story model.

For more investigation, several static pushover analyses were conducted. The base shear is drawn versus lateral roof displacement in Figure 13. It can be seen that the difference between the capacity of the fixed base model and soil structure model is highest for the 8-story model while FS=2. To study the situation, the moment capacities of the systems at the base are calculated. In Figure 13(d), (e) and (f), the theoretical values of the moment capacity of the foundation are estimated using the ASCE-41[41] formulation mentioned in Eq.9.

$$M_{CE} = \frac{L_f P_{UD}}{2} \left(1 - \frac{q}{q_c}\right) \quad (9)$$

Where P_{UD} is the expected vertical load on the soil caused by gravity and seismic loads, q is vertical bearing stress, L_f is the length of footing and q_c is expected bearing capacity. The moment capacities for the fixed base model (M_{FB}) and the soil-structure model (M_{SSI}) are estimated as follows:

$$M_{FB} = (V_U)_{FB} h_e \quad (10-a)$$

$$M_{SSI} = (V_U)_{SSI} h_e \quad (10-b)$$

Where $(V_U)_{FB}$ and $(V_U)_{SSI}$ are ultimate base shear obtained from static pushover curve for the fixed base model and soil-structure model respectively. h_e is the effective height of the system.

In Figure 13(d), (e) and (f), it can be seen that if the theoretical capacity of the foundation is larger than the capacity of the fixed base models, then the capacity of the soil-structure systems is almost the same as the capacity of the fixed base frames (this is true for FS=3 and 5). So, the performance of the soil-structure systems is almost identical to fixed-base models under earthquakes as shown in Figure 13. This is because according to Table 1 the value of the period ratio (T_{SSI}/T_{fix}) is near unity so the effect of elastic soil-structure interaction can be neglected. Thus, without considerable nonlinear behavior at the soil-foundation interface, the responses of the fixed base model and soil-structure model will be almost similar.

On the other hand, if the theoretical capacity of the foundation is smaller than the capacity of the fixed base models, then the capacity of the soil-structure systems is the same as the theoretical capacity of the foundation (this is true for FS=2). In this situation, the ductility of the superstructure can be reduced (in comparison with the corresponding fixed base model) because of the dissipation of seismic input energy as a result of nonlinear behavior in the soil beneath the foundation.

In other words, foundation and structure behave in series, which means that the capacity of the soil-structure systems will be dominated by the smallest capacity value. Consequently, the moment ratio (R_M) is proposed in Equation 11 to evaluate the influence of soil-foundation nonlinearity. If $R_M > 1$, the structure will not be influenced by soil-foundation nonlinearity.

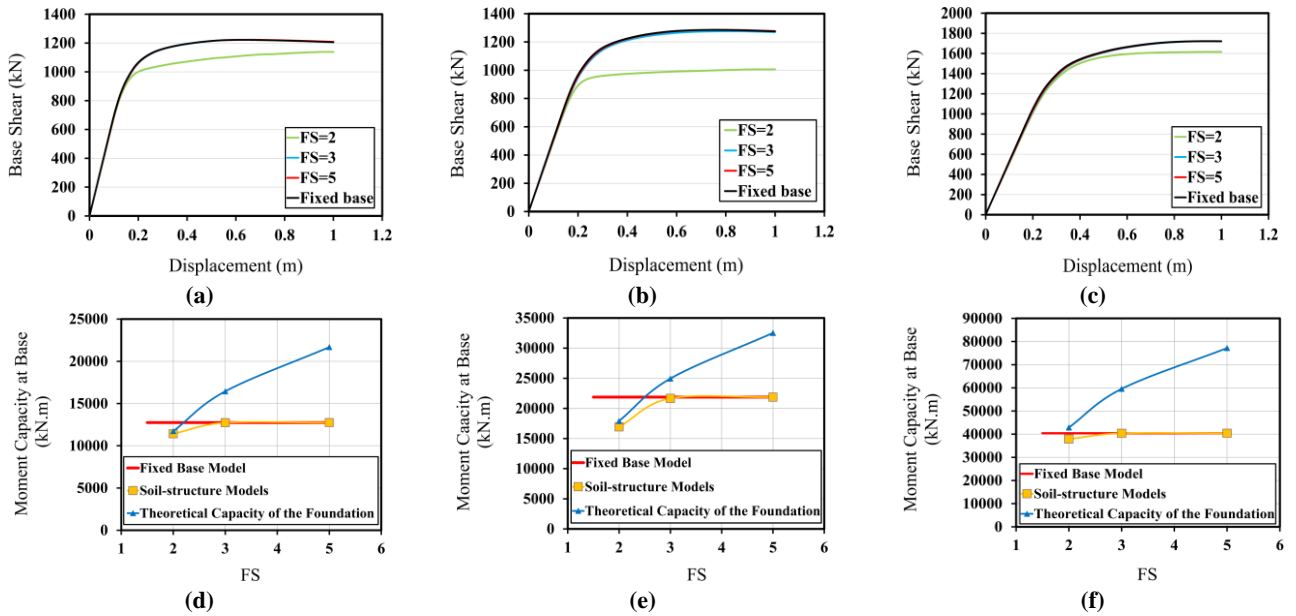


Fig. 13: Pushover curves for 5, 8 and 11-story considering various conditions at base ($V_s=365\text{m/s}$): (a) Pushover curves for the 5-story model; (b) Pushover curves for the 8-story model; (c) Pushover curves for the 11-story model; (d) Moment capacities at base of the 5-story model; (e) Moment capacities at base of the 8-story model; (f) Moment capacities at base of the 11-story model.

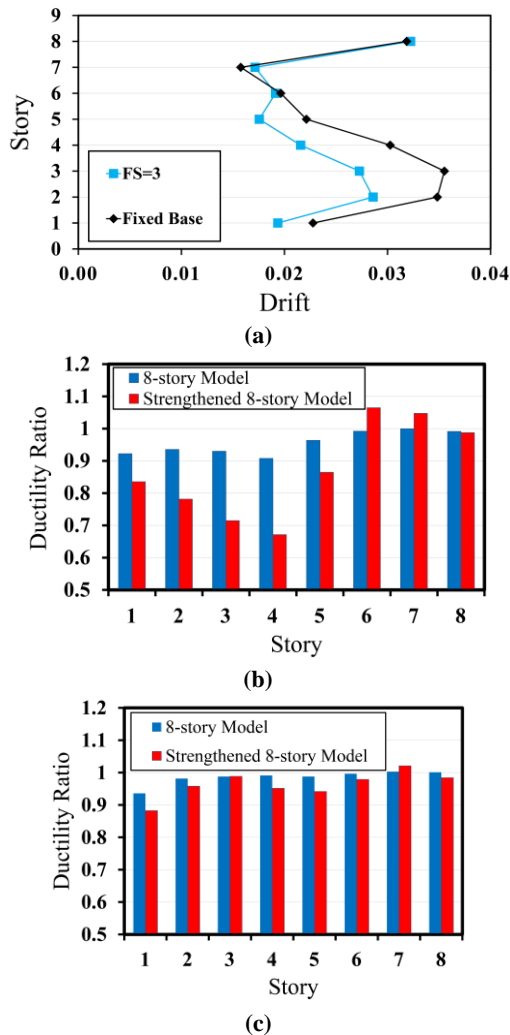


Fig. 14: Drift and ductility ratios for strengthened 8-story model ($V_s=365\text{m/s}$): (a) Drift of Strengthened 8-story model; (b) Ductility ratios of beams; (c) Ductility ratios of columns.

The effect of soil-foundation nonlinearity on the performance of the structure increases by decreasing R_M from one to lower values.

$$R_M = \frac{M_{CE}}{M_{FB}} \tag{11}$$

The 8-story building model with FS=3 is selected to examine the variation of R_M . In Figures 7 and 8, it can be seen that the influence of nonlinearity at the soil-foundation interface is not significant on the performance of the 8-story building model with FS=3, and the system experiences a noticeable level of ductility demands. As shown in Figure 13(e) it can be seen that the capacity of the foundation is larger than the capacity of the structure. To evaluate R_M or in other words, the idea that suggests the foundation and structure behave in series, the capacity of the structure, and the intensities of the artificial records were increased by 20 percent. It should be noted that the excitation was intensified just to acquire a noticeable nonlinear behavior in the structure after increasing its capacity. In Figure 14, it can be seen that the effectiveness of soil-foundation nonlinearity on the improvement of the performance of the super-structure is increased while both the capacity of the structure and the intensity of the excitation have been increased. This is because the moment capacity of the structure has become larger than the moment capacity of the foundation by strengthening.

It should be noted that, although the FS is considered to remain equal to 3, R_M is reduced to take a value smaller than 1. As a result, the capacity of the foundation has become determinative in the soil-structure system which leads to the formation of noticeable nonlinear behavior at the soil-

foundation interface. Consequently, the input energy of excitation can be dissipated more before entering the super-structure.

5. Conclusions

The main idea in this study was to improve the seismic performance of structures by increasing nonlinear behavior at the soil-foundation interface. For this purpose, three moment steel frames were designed and placed on different foundation types. Nonlinear behavior was assigned to the soil-foundation interface. Different shear wave velocities and safety factors were considered for the soil-foundation systems. Then, the soil-structures systems were subjected to seismic ground motions. The ductility demands of beams, columns, and story drifts were assessed. The results showed that decreasing the shear wave velocity of the soil has no significant effect on the performance of the structures while the soil behaves elastically, because the period ratio of models was increased slightly. It was seen that by decreasing the FS to 2, the ductility demands in elements and drift of stories can be reduced significantly. Studying the details of the results showed that decreasing the safety factor does not have the same influence on the performance of different structures. So, a new parameter called the Moment ratio (R_M) is proposed and the variation of the seismic demands of structures as a result of the soil-foundation nonlinearity could be described by it. If the moment capacity of the soil-foundation system is smaller than the super-structure ($R_M < 1$), nonlinear behavior at the soil-foundation interface dissipates the input energy which is conducted to the super-structure. Consequently, the performance of the super-structure can be improved. If the moment capacity of the soil-foundation is larger than the super-structure ($R_M > 1$), then the soil-foundation system does not experience a high level of nonlinearity. One of the unintended consequences of nonlinear behavior at the soil-foundation interface is larger permanent settlements beneath the foundations. It was indicated that the permanent settlements of the foundations were smaller than the values limited by seismic codes while FS is considered to be equal to 2. Finally, it is proposed that considering lower values for the FS instead of common values greater than 3 and reducing the moment capacity of soil-foundation in comparison with the moment capacity of the super-structure, can lead to improvement in the performance of the super-structure.

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