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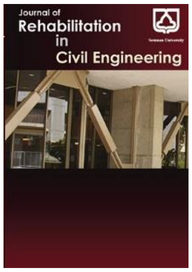

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Seismic Evaluation of Flexible-Base Low-Rise Steel Frames Using Beam-On-Nonlinear-Winkler-Foundation Modeling of Shallow Footings

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ABSTRACT

Recent studies have revealed that the influences of Soil-Structure Interaction (SSI) can be detrimental to the seismic behavior of structure, and hence ignoring this phenomenon in analysis and design may result in to an un-conservative design. The aim of this paper is to quantify the effects of nonlinear SSI on the seismic response of a low-rise special moment frame subjected to a family of ground motions with three hazard levels. To this end, seismic behavior of a 5-story special steel frame founded on linear and nonlinear flexible-base foundations are compared to the conventional fixed-base frame counterpart. The well-known Beam-on-Nonlinear-Winkler-Foundation (BNWF) approach is employed to model nonlinear soil-shallow foundation. Nonlinear static and time history dynamic analyses were conducted applying the OPENSEES platform in order to inquire the effect of modeling and ground motion parameters on their seismic performance. The results manifested some degrees of inaccuracy in the fixed-base and linear SSI assumptions when compared to its nonlinear flexible-base counterpart. Moreover, it is also found that disregarding the foundation flexibility effect may lead to over prediction of the inter-story drift, force and ductility demands of the low-rise steel structure.

1. Introduction

During an earthquake event, deformation at the foundation level occurs as a consequence of the superstructure vibration associated with the inertia forces in the structure. Pursuant to many previous researches, it has been demonstrated that dynamic specifications of a structure which are usually acquired from modal analysis of rigid-base (fixed-base) foundation may differ substantially from the actual response of the

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structure when the influence of soil–structure interaction (SSI) is predominant. Accumulation of maximum capacity and subsequent energy dissipation through a strong ground motion which lead to nonlinear behavior of a soil-foundation interface may alternate the response of a superstructure, i.e., the structure located on the foundation, in several manners. First, foundation movement can elongate the fundamental period of vibration in the structure as a result to its additional flexibility. Second, structural strength demands may be diminished due to the hysteretic energy dissipation, material plasticity and nonlinearity at the soil-foundation interface. And finally, foundation deformations can change the characteristics of input ground motion which is called kinematic soil-structure interaction. Nevertheless, the conventional design practice is still unwilling to deliberate the nonlinear SSI effects by stating two justifications: (1) the SSI effect is beneficial and not consideration of this phenomenon leads to a more conservative design, and (2) there is no approach to reliably account for SSI nonlinear behavior. It is also obvious that the flexibility of the soil beneath the structure results in a decrease in the total shear stiffness of a structure, and consequently results in the natural period of the soil-structure systems is larger than the corresponding fixed-base structures [1-3]. In addition, soil supporting the structure increases damping ratio of the SSI systems as a result of the radiation and inherent damping [3]. Pursuant to the research conducted by Stewart et al. [4], these variations in dynamic characteristics of soil-structure systems can have a remarkable effect on different design decisions, and thus require a precise evaluation. The present seismic design and rehabilitation codes, such as ATC [5], NEHRP [6], FEMA [7], and ASCE [8], take into account SSI influences by accounting for an enlarged effective damping ratio and period, through some modifications in the pseudo acceleration for seismic-resistant design. Nevertheless, the unwillingness or hesitancy to take into account SSI for structural analysis and design is still obvious within the structural engineering community.

Previously, many researches have gone into the evaluation of the SSI effects on the seismic demands of elastic and inelastic response of buildings. Veletsos and Meek [1] and Veletsos and Nair [9] proposed some equations regarding equivalent period and damping of structures because of SSI effects via an equivalent SDOF fixed-base system utilizing an equivalent sway and rocking spring beneath the foundation. Applying a series of elastic-plastic Winkler springs, Chopra and Yim [10] practically modeled SSI phenomenon. Employing finite element discretization of the subsoil, Kim and Roesset [11] also proposed a new idealized SDOF system of a building with a shallow foundation located on soft soil. In general, the research studies performed on SSI systems can be divided into two sections: one, modeling soil surrounding the structures with equivalent elastic model such as cone model proposed by Wolf [12], and two, nonlinear SSI modeling which is more realistic model during moderate and severe earthquake events. For the first case, many studies have been conducted for SDOF and MDOF soil-shallow and embedded-foundations [13-20]. For the case of nonlinear SSI modeling, in some current studies, the phenomenon was modeled via rigorous direct model of finite-element method such as those conducted by Maheshwari and Sarkar [21]; Tabatabaiefar et al. [22] and Hokmabadi et al. [23]; however, most of other researchers preferred advanced discrete lumped-mass models [24-28]. More recently, the nonlinear advanced discrete model of BNWF approach has attracted researchers for SSI analyses as a result to its capability and simplicity for modeling complex SSI phenomenon more realistically. Raychowdhury and Singh [28] applying nonlinear dynamic and static pushover analyses on a 3-story steel moment resisting frame assessed the performance of the structure based on different demand parameters. Marzban et al, [29] employed BNFW approach in order to inspect the effect of nonlinear SSI on structural responses of the earthquake-induced shear wall frames. Using the same SSI model, Vivek and Raychowdhury [30] investigated the effect of nonlinear SSI on dynamic properties of soil-steel structures located on dry sand soils.

This paper made an attempt to examine the effects of nonlinear SSI on the seismic performance of a low-rise special steel moment frame compared to their fixed-base counterparts. The sophisticated nonlinear-Winkler-foundation approach proposed by Raychowdhury and Hutchinson [27] for shallow foundations is applied to inquire the influence of nonlinear SSI on seismic behaviour of low-rise special SMF buildings subjected to a family of far-fault ground motions with three hazard levels.

2. Description of the Low-rise Building Considered in This Study

The buildings contemplated in the study is a 5-story special steel moment-resisting frame building resting on alluvium soil site corresponding to the site classification D, from NEHRP [6]. The considered building has a plane which consists of four bays on each direction as portrayed in Fig. 1. It has a floor plan of 28 m×28 m and four bays with 7.0 m in each of the two horizontal directions. The mass and stiffness are distributed uniformly and non-uniformly in conformity with the height of the structure, respectively. The story height of the building is deliberated as 3.2 m which is a normal height for residential buildings. The dead and live loads of 650 and 200 kgf/m^2, respectively, are considered as gravity loads. The building is designed in accordance to the seismic force distribution specified by the ASCE-7-10 [8] for Zone IV. In addition, the strength and stiffness of the frame members have been selected such that the fundamental periods of vibration that are representative of the conventional existing residential building. In addition, the building pondered in this paper corresponds to strong column-weak beam design philosophy. Fig. 2 illustrates the configuration and properties of their exterior frames. The first three natural frequencies are respectively 1.35, 4.02 and 7.14 Hz. Modified-Clough hysteresis model is applied to represent the load-deformation characteristics of hinges with stiffness degradation [17].

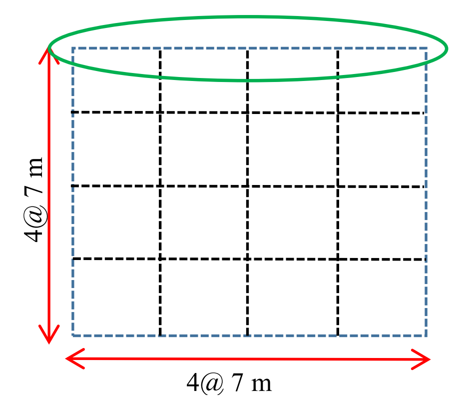

Fig. 1. Floor plan and frame of the building considered in this study.

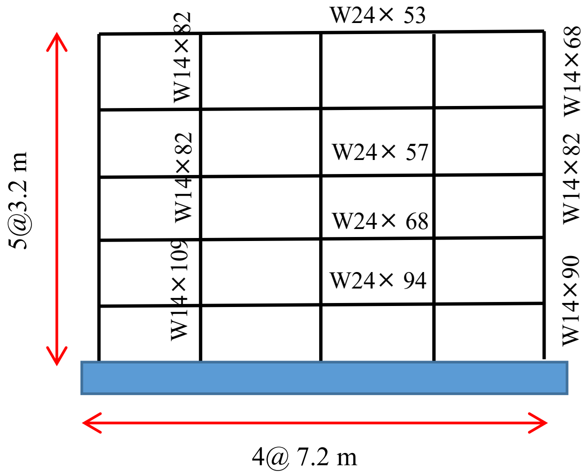

Fig. 2. Elevation of the building showing steel members.

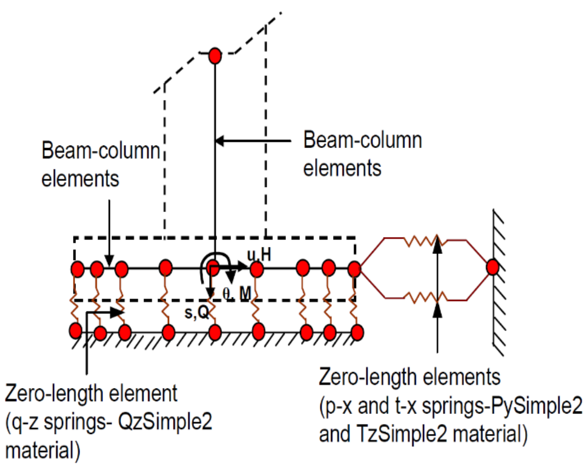

Fig. 3. The schematic diagram of BNWF showing springs with their orientations [32].

3. Nonlinear Soil–Foundation Model

In the current study, the soil-shallow foundation interface is captured employing a practical BNWF concept. In this regard, the interface of the soil-foundation is simulated assuming independent nonlinear spring elements for footing [26-28]. The model is congregated exercising an array of inelastic spring elements as shown in Fig. 3. The BNWF model consists of a beam-column element and a series of sway and vertical zero-length springs, called as q-z, p-x, and t-x springs. The vertical q-z spring elements are set beneath the foundation to incorporate the vertical i.e., uplift and settlement, and rocking resistance of the footing. A non-uniform stiffness arrangement is assigned in accordance with length of the BNWF model. In order to acquire an assumed rocking stiffness of the footing, the ratio of stiffness springs in end to mid region was taken the values of five along the length of ten percent of the foundation length in conformity with the recommendations provided by Harden et al. [31] and the ATC-40 [5]. The vertical springs at the end and mid zones are placed at a spacing of 1% 2% of the footing length, respectively [32]. The initial elastic stiffness and the vertical ultimate capacity are computed based on the expression proposed by Gazetas [33] and Terzaghi [34]. The nominal tension capacity of q-z springs was taken 5% of ultimate capacity. In addition, the two other horizontal springs are deliberated to incorporate the passive and sliding effects of the footing, respectively, as depicted in Fig. 3.

The constitutive hysteretic material behavior used for the *t-x, p-x and q-z* springs elements are provided by nonlinear curves. They were originally proposed by Boulanger [32], which was later rectified by Raychowdhury and Hutchinson [27] for BNWF model through a series of shallow foundation tests. Each spring material of BNWF model is composed of an initial elastic portion with initial elastic stiffness (k_{in}). In the elastic section of a *q-z* material, the momentary load q is supposed to be proportional linearly with the momentary displacement z:

$$q = k_{in} z \tag{1}$$

The range of the elastic zone is defined by:

$$q_0 = C_r q_{ult} \tag{2}$$

Where q_0 and q_{ult} are the yield and ultimate loads, respectively. C_r is a parameter that controls the elastic range portion. For the post-yield part, the backbone curve is designated by:

$$q = q_{ult} - (q_{ult} - q_0)\left[\frac{cz_{50}}{cz_{50} - \left|z^p - z_0^p\right|}\right]^n \qquad (3)$$

where the parameters of z_{50}, z_0^p , and z^p are the displacements, ultimate strength, yield point, and any point load, respectively. Moreover, *c* and *n* are the constant parameters effects that are defined in Ref. [27]. It is noteworthy to mention that the expressions associated with *t-x and p-x* elements are identical to Eqs. (1) to (3), in which the parametres *n*, *c*, and C_r are varied in order to control the general form of the curve. In addition, the *q-z* material has a decreased capacity in the tension part, permitting the foundation to uplift taken into consideration of contact with the soil bellow footing throughout a rocking. Besides, the *p-x* material is described by a pinched hysteretic behavior, in which the *t-x* material is described by a large initial stiffness and a broad hysteretic loop (see Ref. [26-28]). Pursuant to Eqs. (1) to (3), the response of nonlinear spring members is mainly dependent on q_{ult} and k_{in} that can be associated to the z_{50} as follows:

$$z_{50} = F_k \frac{q_{ult}}{k_{in}} \qquad (4)$$

where F_k is a controlling factor computed for shallow foundation tests. The footing bearing capacity and the related parameters are acquire from Terzaghi [34] with key governing factors after Meyerhof [35] as expressed in Eqs. (5) to (8):

$$q_{ult} = cN_c F_{cs} F_{cd} F_{ci} + \gamma D_f N_q F_{qs} F_{qd} F_{qi} + 0.5\gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i} \qquad (5)$$

where, *c*= the soil cohesion, γ = the unit weight of soil, D_f = the depth of embedment, and *B* is the width of footing. F_{cs}, F_{qs}, F_{gs} are the shape factors, F_{cd}, F_{qd}, F_{gd} are the depth factors and F_{ci}, F_{qi} and F_{gi} are the inclination factors, which are computed after Meyerhof [34]. N_γ , N_q, and N_c, computed by the following equations:

$$N_q = \tan^2(45^o + \frac{\varphi}{2})e^{\pi \tan\varphi} \qquad (6)$$

$$N_c = (N_q - 1)\cot\varphi \qquad (7)$$

$$N_\gamma = (N_q - 1)\tan(1.4\varphi) \qquad (8)$$

where φ is the soil friction angle. For the *p-x* spring element, p_{ult} , defined as passive earth pressure divided by unit length of foundation, is calculated though the total passive resisting force that acts on the embedded footing front side. In addition, the passive resisting force for homogeneous backfill against the footing can be computed as follows [27]:

$$P_{ult} = 0.5\gamma D_f^2 K_P \qquad (9)$$

where K_P is the passive earth pressure coefficient. Further, the total frictional resistance, t_{ult}, is computed through the Eq. (10) [27]:

$$t_{ult} = W_g \tan\delta + A_b c \qquad (10)$$

in which W_g is a vertical force exerted on the foundation, δ is the angle of friction between the soil and foundation (basically altering from $1/3\varphi$ to $2/3\varphi$), and A_b is the footing area in contact with soil and is equal to L times B.

According to the expression proposed by Gazetas [33], the initial lateral stiffness, k_h and vertical stiffness, k_v, of the footing in the elastic range are derived by the Eqs. (11) and (12).

$$K_v = \frac{GL}{1-\nu}\left[0.73+1.54\left(\frac{B}{L}\right)^{0.75}\right] \tag{11}$$

$$K_h = \frac{GL}{2-\nu}\left[2+2.5\left(\frac{B}{L}\right)^{0.85}\right] \tag{12}$$

where G and v are the soil shear modulus of elasticity and Poisson's ratio; and B/L is the ratio of footing width to footing length. The k_p in the nonlinear area of the backbone curves is computed by rearranging Eq. (13) as follows:

$$k_p = n(q_{ult}-q_0)\left[\frac{(cz_{50})^n}{(cz_{50}-z_0+z)^{n+1}}\right] \tag{13}$$

Previous studies have reported that the BNWF SSI model has a capability of capturing the governing specifications of a shallow footing behavior. It was also demonstrated that the nonlinear SSI model employed in this study can reliably predict various seismic demands of footing including initial and post-yield stiffness of moment-curvature plot, maximum moment and shear, transient and residual sliding, settlement and rotation as well as overall shape of the shear-sliding and moment-rotation loops [27-30,36].

In this paper, the shallow foundation is assumed to be on stiff silty sand corresponding to the site class D based on NEHRP [6], with unit weight of 20 kN/m^3, cohesion of 70 kN/m^2, Poisson's ratio (ν) of 0.4. The foundations are designed for 6 different values of the static vertical safety factor equal to 3, 3.5, 4, 4.5, 5, 5.5, implying its bearing capacity is from 3 to 5.5 times of the applied vertical load. The non-uniformly distributed vertical springs are modeled accompanying the length of the foundation to simulate the foundation rocking stiffness. The vertical springs are distributed at a spacing of 1 and 2% at the end and mid regions of the footing length. As aforementioned, pursuant to the recommendations of ATC-40 [5] and Harden and Hutchinson [31], stiffness intensity ratio at the edges of the footing is larger than the stiffness of middle region.

4. Selecting of Earthquake Ground Motions

For nonlinear analyses, 22 strong earthquake acceleration record were compiled. These earthquake ground motions have been selected based on the following criteria: (a) They exclude the near-fault ground motion characteristic such as pulse-type and forward directivity effects; (b) They have no long duration characteristics. The selected earthquake ground motions have moment magnitude larger than 6.5, and the distance to the fault rupture between 13 km and 40 km. These ground motions are recorded on deposits that correspond to IBC-2015 site class D, which is approximately similar to the soil type III of the Iranian seismic code of practice, Standard No. 2800 [37] . Additional criteria such as magnitude of earthquake, site distance to source, and ground motion characteristics have been adapted to further refine the data of earthquake records to be applied in the study. A description of these additional criteria is presented in Table 1. The selected ground motions are scaled based on ACSE-7-10 [8] for three ground motion hazard levels of 50/50, 10/50, and 2/50 as will be discussed in the next sections.

Table 1. List of earthquake ground motions recorded on site class D.

Event	M_w.	Station Name	Soil Type	R (Km)	PGA (g)	PGV (cm/s)
Loma Prieta	6.9	Agnews State Hospital	D	28.2	0.172	26
Loma Prieta	6.9	Capitola	D	14.5	0.443	29.3
Loma Prieta	6.9	Gilroy Array #3	D	14.4	0.367	44.7
Loma Prieta	6.9	Gilroy Array #4	D	16.1	0.212	37.9
Loma Prieta	6.9	Gilroy Array #7	D	24.7	0.226	16.4
Loma Prieta	6.9	Hollister City Hall	D	28.2	0.247	38.5
Loma Prieta	6.9	Sunnyvale—Colton Ave.	D	28.8	0.207	37.3
San Fernando	6.6	LA—Hollywood Stor Lot	D	21.2	0.174	14.9
Superstition Hills	6.7	Brawley	D	14	0.156	13.9
Superstition Hills	6.7	El Centro Imp. Co. Cent	D	21	0.358	46.4
Superstition Hills	6.7	Plaster City	D	17.2	0.186	20.6
Northridge	6.7	LA—Centinela St.	D	30.9	0.322	22.9
Northridge	6.7	Canoga Park—Topanga Can.	D	15.8	0.42	60.8
Northridge	6.7	LA—N Faring Rd.	D	23.9	0.273	15.8
Northridge	6.7	LA—Fletcher Dr.	D	29.5	0.24	26.2
Northridge	6.7	LA—Hollywood Stor FF	D	25.5	0.231	18.3
Northridge	6.7	Lake Hughes #1	D	36.3	0.087	9.4
Northridge	6.7	Leona Valley #2	D	37.7	0.063	7.2
Imperial Valley	6.5	El Centro Array #1	D	15.5	0.139	38.1
Imperial Valley	6.5	El Centro Array #12	D	18.2	0.116	16
Imperial Valley	6.5	El Centro Array #13	D	21.9	0.139	21.8
Imperial Valley	6.5	Chihuahua	D	28.7	0.27	13

5. Numerical Analysis

The SMF prototype was analyzed through nonlinear dynamic time history method subjected to 22 earthquake ground motions listed in Table 1. In order to conduct the transient analysis, Newmark-β method was employed with problem parameters of 0.25 and 0.5. Structural damping is modelled based on Rayleigh type damping model. In this regard, five percent of critical damping was deliberated to the first and second modes of vibration. Furthermore, in order to solve the nonlinear equations, the modified Newton–Raphson algorithm taking into consideration of 40 iterations and a convergence tolerance of 1e-9 was applied [38]. The results of the all the nonlinear analyses are provided in the upcoming section. To examine the effects of soil nonlinearity of different seismic demand parameters, herein, nonlinear dynamic analyses are carried out for 1188 fixedand SSI models. They include 6 values of vertical factor of safety (VFS= 3, 3.5, 4, 4.5, 5, 5.5), 3 types of foundation i.e., fixed-base model, linear and nonlinear SSI subjected to 22 earthquake ground motions corresponding to three ground motion hazard levels of 50/50, 10/50, and 2/50. Note that the 50/50, 10/50, and 2/50 ground motion hazard levels are defined as that corresponding to 50%, 10% and 2% probability of exceedance of a target earthquake intensity measure during fifty years, respectively.

6. Results and Discussion

In order to assess seismic behavior of building structures contemplating linear and nonlinear SSI effect, a parametric study is conducted. Before performing nonlinear dynamic analyses, nonlinear static pushover and eigenvalue analyses should be performed to compute the basic information required for understanding the behavior of the nonlinear soil-structure system. In seismic-resistant design of structures, fundamental period of vibration is one of the major parameters to estimate the structural and nonstructural seismic demands. Modern building codes generally apply the period elongation or so-called period ratio defined as the fundamental period of vibration of fixed-base system normalized to that of flexible-base one to evaluate the detrimental or beneficial influences SSI on the seismic response.

Here, through a modal analysis, periods of the fixed-base and flexible-base models are determined for different values of vertical factor of safety (VFS) ranging from 3 to 5.5. The results reveal that the period ratios (i.e., SSI period normalized to the fixed-base period) change from 1.11 to 1.17 as VFS reduces from 5.5 to 3. The variation are from 1.023 to 1.067 and 1.015 to 1.03 for the second and third modes, respectively. Moreover, it is also found that regardless of linear or nonlinear assumptions of soil, SSI can affect the fundamental period of vibration significantly; whereas, it insignificantly influences on the period of higher modes.

6.1 Influence of Nonlinear SSI on Total Shear Strength

In force-based design method, base shear strength is mainly selected as one of the key response parameters in seismic design practice. In this section, in order to examine the effects of soil nonlinearity on total base shear strength, nonlinear dynamic analyses are carried out for 1188 fixed- base and flexible base models described in the previous section. Figs. 4(a) through (c) provide the mean ratios of base shear strength (*BSR*) defined as the base shear in fixed-base systems to that in corresponding linear and nonlinear SSI systems as follows:

$$BSR = \frac{V_{FB}}{V_{SSI}} \tag{14}$$

where V_{FB} is fixed-base shear strength and V_{SSI} is the corresponding base shear strengths for linear and nonlinear SSI models. The results are depicted as mean values of 22 earthquakes for three aforementioned hazard levels. As it is evident, except for the low hazard level of 50/50, the linear-base shear strength demand is a little larger (about 3%)

than the fixed-base shear strength demand for all ranges of VFS. Notwithstanding, for the nonlinear SSI case, the BSR value is always larger than unity, indicating a significant reduction of the shear strength demand transpires in nonlinear SSI when compared to that in fixed-base and linear SSI models. Further, it is also observed that unlike to the linear systems, the reduction is considerably dependent on the vertical factor of safety. Therefore, deliberating the fact that when a structure is subjected to a ground motions ensemble with higher intensities (e.g., 10/50 and 2/50), the soil beneath the structure becomes nonlinear, it is computed that modeling the soil-foundation interface as fixed or linear elastic would result in overestimation of base shear strength. The phenomenon is more pronounced for the case of larger VFS, implying the possibility of more reduction (up to 21%) in base shear strength demands because of nonlinear behavior of assigned springs at the soil-foundation interface.

6.2 Effect of Nonlinear SSI on Absolute Base Moment

Similar to the results allocated for base shear strength, Figs. 4(d) through (f) display the mean ratios of base moment (*BSR*) defined as the maximum absolute base moment demand in fixed-base system to that in corresponding linear or nonlinear SSI system as follows:

$$BMR = \frac{M_{FB}}{M_{SSI}} \tag{15}$$

where M_{FB} is the maximum absolute fixed-base moment demand and M_{SSI} is the corresponding value for linear and nonlinear SSI models. The mean ratios of 22 ground motion listed in Table 1 are computed for ground motions of hazard levels 50/50, 10/50, and 2/50, and are illustrated verses the vertical static factor of safety (VFS) of the foundation. It is found that the base moment demand for the fixed-base model is up to 15% (i.e., 50/50 hazard level) greater than that for elastic flexible base one. The ratio boosts even up to 60% for the case of nonlinear foundation interface with higher ground motion intensities. It can also be observed that the ratio is not sensitive to the variation of vertical factor of safety in the case of elastic foundation; whereas, it is remarkably influenced by the foundation conformity when the nonlinearity in the soil-foundation interface is taken into account. The phenomenon is more intensified by increasing the ground motion intensity.

6.3 Effect of Nonlinear SSI on Roof and Maximum Drift Demands

In performance-based seismic design, inter-story drift demand is the most significant response parameter in seismic design application. Figs. 5(a) and (b) devote the influence of the flexibility and nonlinearity of the foundation on the maximum inter-story drift of the building for the different hazard levels (i.e., 50/50, 10/10 and 2/50). For better understanding, the ratio of maximum drift demand defined as the maximum inter-story drift demand in the fixed-based systems normalized to that in linear and nonlinear SSI systems are computed and the mean results are plotted for different hazard levels. As it is evident from Fig. 5(a), the drift demand in linear SSI systems is generally greater than fixed-base systems, and it increases from 6% to 11% with boosting the ground motion intensity.

Conversely, the maximum drift demands present a diminishing trend when base nonlinearity is to be deliberated as revealed in Fig. 5(b). The reduction varies from 7% to 35% for low to high hazard levels, respectively. This signifies that, in average, the peak drift demand can considerably alternate when nonlinear SSI is considered. Therefore, considering the fact that the maximum inter-story drift demand is a key parameter in performance-based seismic design, it is most possible that the member elements were designed over-conservatively when the effect of nonlinear SSI is neglected. In a similar trend, Figs. 6(a) and (b) also manifest the mean roof drift ratio defined as the maximum roof drift demand in the fixed-based systems normalized to that in linear and nonlinear SSI systems for the different hazard levels. It can be observed that depending on the hazard level, the elastic base roof drift can be greater or lower than the corresponding fixed-base system. In the case 50/50, the roof drift of fixed-base model is in average 10% lower than elastic base foundation while it is 7% greater than that for severe ground motion intensity, 2/50. However, the trend for nonlinear SSI systems is similar to that described for the maximum inter-story drift. In fact, the average roof drift demand in fixed-base system is up to 21% greater than the nonlinear flexible-base one in 2/50 hazard level.

6.4 Effect of Nonlinear SSI on Inter-Story Ductility Demand

In performance-based seismic design framework, inter-story ductility demand is also a critical parameter since it is indirectly related to the energy dissipated trough inelastic behavior of structural members. In this section, the maximum inter-story ductility demand ratios of fixed-base to those in flexible-base systems for two values of VFS= 3 and 5 are computed for different hazard levels and the mean results are provided in Fig. 7. It is noteworthy to mention that the maximum ductility demand is the peak value among all stories defined as the ratio of maximum absolute drift demand obtained from nonlinear dynamic analysis to the yield drift acquired from the static pushover analysis. As it is evident, irrespective of the base condition, the ductility demands in fixed-base systems are, in average, larger than flexible base systems. In addition, the ductility demand diminishes more when foundation flexibility is modeled nonlinearly. The ratio is higher for lower intensity motions, yet decreases for higher intensities. It is essential to emphasize that the nonlinear SSI case demonstrates the ductility demand to be considerably lower than fixed-base systems for all hazard levels, indicating that neglecting the nonlinear SSI would results in a significant overestimation in the ductility demand of structural members. Considering the fact that for sever ground motions the soil beneath the foundation will most probably behave nonlinearly, it is, therefore, essential to incorporate modelling the soil-foundation interface with more realistic model such as the one deliberated in this study for preventing the overdesign of structural members.

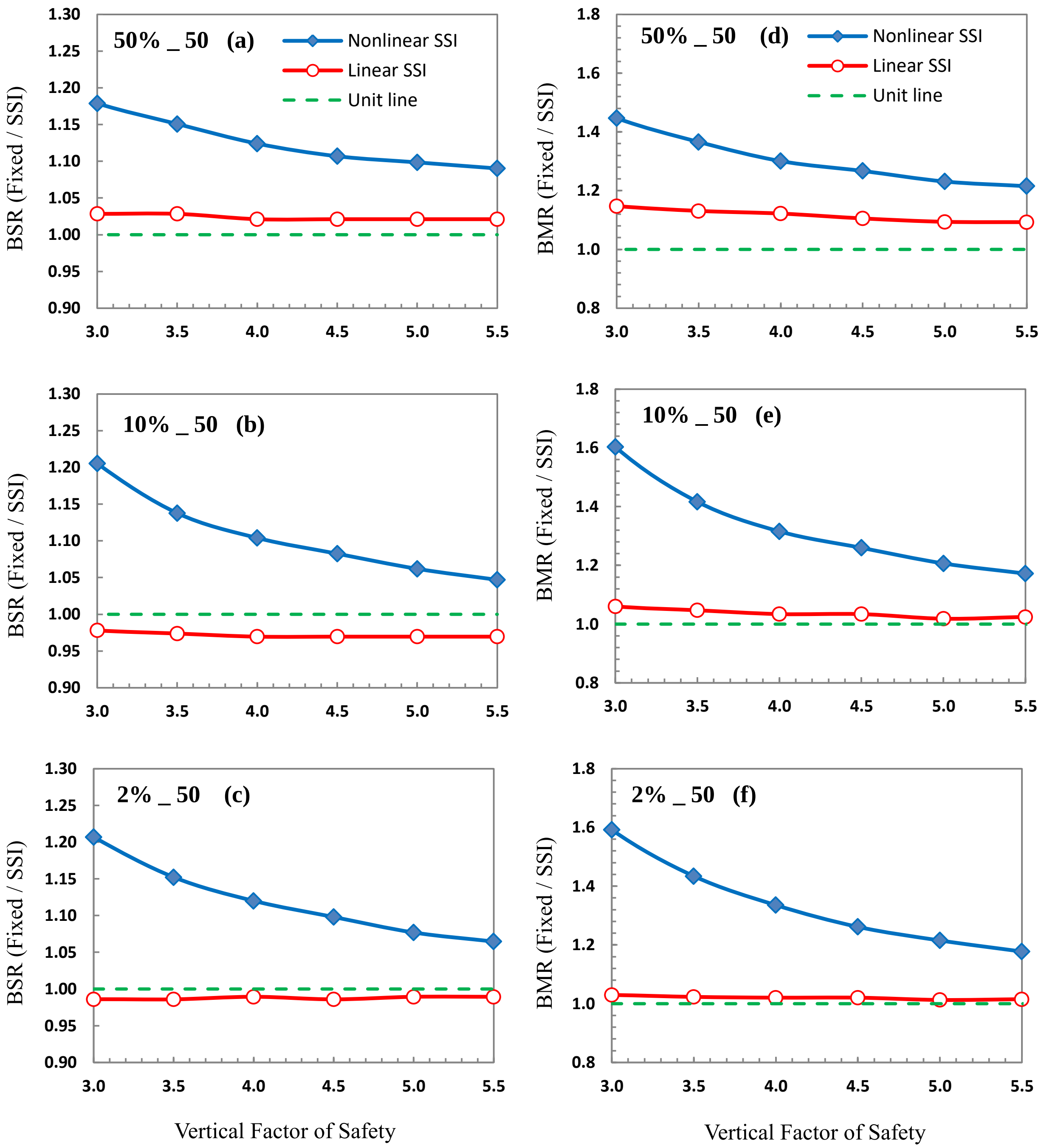

Fig. 4. The mean ratios of base shear (left) and base moment (right) demands of fixed-base models to those of corresponding flexible- base models for different hazard levels.

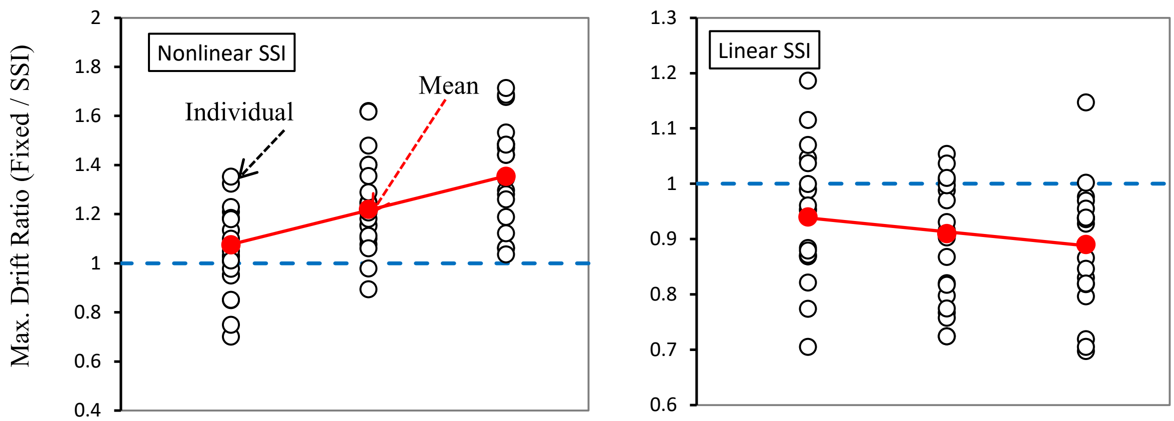

Fig. 5. The mean ratios of maximum drift demands of fixed-base models to those of corresponding nonlinear (lest) and linear (right) flexible-base models for different hazard levels (22 earthquakes)

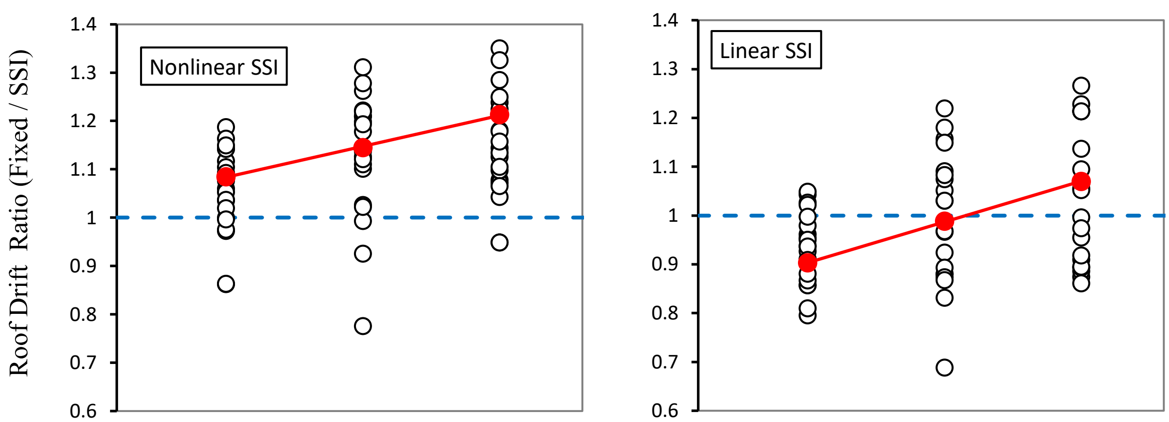

Fig. 6. The mean ratios of roof drift demands of fixed-base models to those of corresponding nonlinear (lest) and linear (right) flexible-base models for different hazard levels (22 earthquakes)

7. Concluding Remark

Nonlinear dynamic analyses are carried out in order to inquire the nonlinearity rule of soil-foundation interface on the earthquake demand parameters of a low-rise special SMRF, crucial for displacement-based design and force-based approaches, in terms of base shear strength, base moment, maximum and roof inter-story drift as well as displacement ductility demand for different levels of hazard subjected to a family of 22 ground motions. To compare the results the mean ratio of above-mentioned demand parameters for fixed-base systems normalized to those for linear and nonlinear SSI systems are computed for 5 values of vertical factor of safety. To this end, Beam-on-Nonlinear-Winkler-Foundation model (BNWF) for shallow footings is adopted. The following conclusions are summarized from this study.

- The ratios of period (i.e., SSI period normalized to fixed-base period) alternate from 1.11 to 1.17 as vertical factor of safety reduces from 5.5 to 3. The variations are from 1.023 to 1.067 and 1.015 to 1.03 for the second and third modes, respectively, which can be contemplated negligible compared to the fundamental period elongation.

- For the nonlinear SSI case, the BSR and BMR values are generally larger than unity, indicating a significant reduction of the shear strength and moment demand transpires in nonlinear SSI when compared to those in fixed-base a linear SSI models.

- The drift demand in linear SSI systems is generally greater than fixed-base systems such that it increases up to 11% for severe ground motion intensity. Conversely, the maximum drift demands reduces up to 35% when base nonlinearity is to be considered.

- Depending on the hazard level, the elastic roof drift can be greater or lower than the corresponding fixed-base system. In the case 50/50, the roof drift of fixed-base model is in average 10% lower than elastic base foundation while it is 7% greater than that for severe ground motion intensity, 2/50.

- Regardless of the base condition, the ductility demands in fixed-base systems are, in average, larger than flexible base systems. In addition, the ductility demand diminishes more when foundation flexibility is modeled nonlinearly, demonstrating that neglecting the nonlinear SSI would results in a significant overestimation in the ductility demand of structural members.

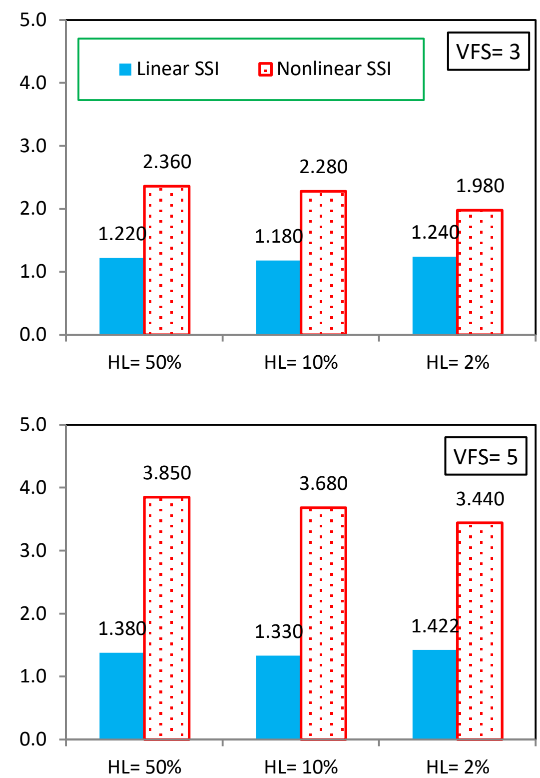

Fig. 7: The mean ratios of ductility demands of fixed-base models to those of corresponding nonlinear and linear flexible-base models for different hazard levels.

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