Assessment of the safety margin in the seismic design of foundations based on ASCE/SEI 41-06 standard

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ABSTRACT

The seismic linear demand of structures is usually reduced by employing a force-reduction factor in most of force-based seismic regulations. The current force reduction factors in ASCE/SEI 41-06 standard, results in conservative design in the case of foundations when compared to the conventional design regulations e.g. IBC 2000. The aim of the current paper is to evaluate the influence of the Soil-Foundation-Structure-Interaction (SFSI) effects on the force-reduction factor based on ASCE/SEI 41-06 standard. Therefore, a comparison has been made between the results of the nonlinear response history analysis of a set of 3, 6, 10 and 15-storeyed concrete 2-D frames with the result of the equivalent linear static approach. The results show that the equivalent linear static approach load combinations, in the case of foundations, can lead to conservative designs. Finally a set of new force-reduction factor has been calculated in order to cope with this problem.

Key words: Seismic Design/Rehabilitation of Foundations, Force Reduction Factor, Linear Static Procedure, Nonlinear Dynamic Procedure, ASCE/SEI 41-06 Standard.

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1. Introduction

The material nonlinear behaviour of different elements usually dissipates significant amount of seismic input energy. Therefore, on the basis of strength based codes, designers are allowed to divide the elastic force response by a force delivery reduction factor, named J factor in ASCE/SEI -06 standard. The force reduction factor is defined based on several bases including: experimental and analytical aspects, engineering judgement and structure performance in previous earthquakes [1], [2]. As this factor is different in various regulations, it worth focusing on the subject in order to comprehensively define the force reduction factor. That is, the current research is aimed to calculate the force reduction factor, by comparing the results of equivalent linear static procedure with nonlinear response history procedure. A set of concrete moment resisting frames with shear walls were selected for the purpose of comparison. The results show that the proposed load combinations in ASCE/SEI 41-06 standard highly overestimate the seismic demand for the force controlled elements, specifically in the case of foundations.

2. Force delivery factor in ASCE/SEI 41-06 standard

According to 3.2.10 in ASCE/SEI 41-06 standard, all the vertical elements, which are parts of lateral load resisting system, shall be designed by taking the overturning moments into account [3]. The implemented seismic force, which is proposed to be used in the linear static procedure, is usually remarkable compared with the force that is usually recommended by design codes. This force is only reduced with the force delivery factor (J factor). Additionally, the force controlled elements actions are checked with the seismic force that is reduced with J factor. However, this factor is set to be between 1 and 2. For example, J factor is equal to unity in the case of immediate occupancy performance level which is also the case in the current research. The most important consequence is that the seismic demand for the elements, which are controlled with the force action, is remarkable which results in conservative design compared with past design regulations. To cope with this problem, a set of concrete 2-D frames were considered in order to comprehensively investigate the ability of new factors in estimating seismic demands on foundations. The details are discussed in the following sections.

3. Test Structures and analytical models

A set of 2-D concrete moment resisting frames with shear wall, containing 3, 6, 10 and 15-storeyed frames, were designed according to FEMA450 static linear guidelines [4], as schematically shown in Figure 1. Three types of soil, consisting hard, medium and soft, were taken into account introduced through site classes B, C and D. The shear walls were considered in the design procedure since SFSI effects are significant in the case of stiff structures [5]. The storey height and the bay length are, respectively, equal to 350 and 600 cm as seen in Figure 1. The shear wall thickness is 25 cm for the 3 and 6 storeyed frames and it is equal to 30 cm for 10 and 15-storeyed frames [5], [6], [7].

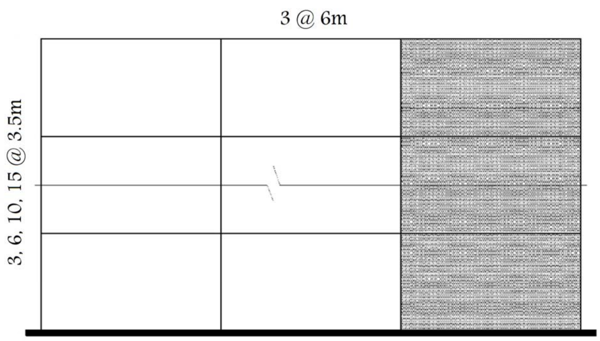

Figure 1. Elevation of the considered frames.

With identified category and shear wave velocity for the site classes, satisfactory values were estimated to represent their design parameters according to several well-known geotechnical references [8], [9], [10]. The gravity loads were selected based on values typically employed in engineering practice. Therefore, 3, 6, 10 and 15-storeyed frames had masses equal to 307, 640, 927 and 1511 tons, respectively. Further, equivalent lateral design forces were determined according to FEMA450 guidelines i.e. the design spectrum for each site class was derived as seen in figure 2 and the corresponding design base shears were calculated [4]. Both of gravity and seismic loads were later imposed on the frames according to the counteractive load combinations of ASCE/SEI7-05 standard [3]. (the reader is referred to Section 12.4.2.3 of ASCE/SEI7-05 for further details.)

The given frames were designed as special frames based on FEMA450 guidelines. According to these guidelines, 15-storeyed frames should be designed considering dual lateral resisting systems. The gravity loads for the designed 3, 6, 10 and 15- storeyed frames are, respectively, shown in Table 1 to Table 4. The strip footings of width 2.0-4.6 m, length 19.6-20.0 m and fixed height 1.0 m, were designed for all frames. Subsequently, the complete numerical models of the Concrete Shear Wall (CSW) frames were constructed through an appropriate assembly of nonlinear shear wall elements and nonlinear beam-column elements.

Table 1. Gravity loads for the designed 3 storeyed frame.

No. of Storey	DL (kg/m)	PL (kg/m)	No . of Bays	L(m)	Load weight (kg)
3	3600	0	3	5.55	53946
1-2	3900	600	3	5.55	67432.5

Table 2. Gravity loads for the designed 6 storeyed frame.

No. of Storey	DL (kg/m)	PL (kg/m)	No. of Bays	L(m)	Load weight (kg)
6	3600	0	3	5.55	53946
Other	3900	600	3	5.55	67432.5

Table 3. Gravity loads for the designed 10 storeyed frame.

No. of Storey	DL (kg/m)	PL (kg/m)	No. of Bays	L(m)	Load weight (kg)
10	2700	0	3	5.55	40459.5
4-9	3000	600	3	5.55	53946
1-3	3000	600	3	5.45	52974

Table 4. Gravity loads for the designed 15-storeyed frame.

No. of Storey	DL (kg/m)	PL (kg/m)	No. of Bays	L(m)	Load weight (kg)
15	2700	0	3	5.55	40459.5
9-14	3000	600	3	5.55	53946
5-8	3000	600	3	5.4	52488
1-4	3000	600	3	5.3	51516

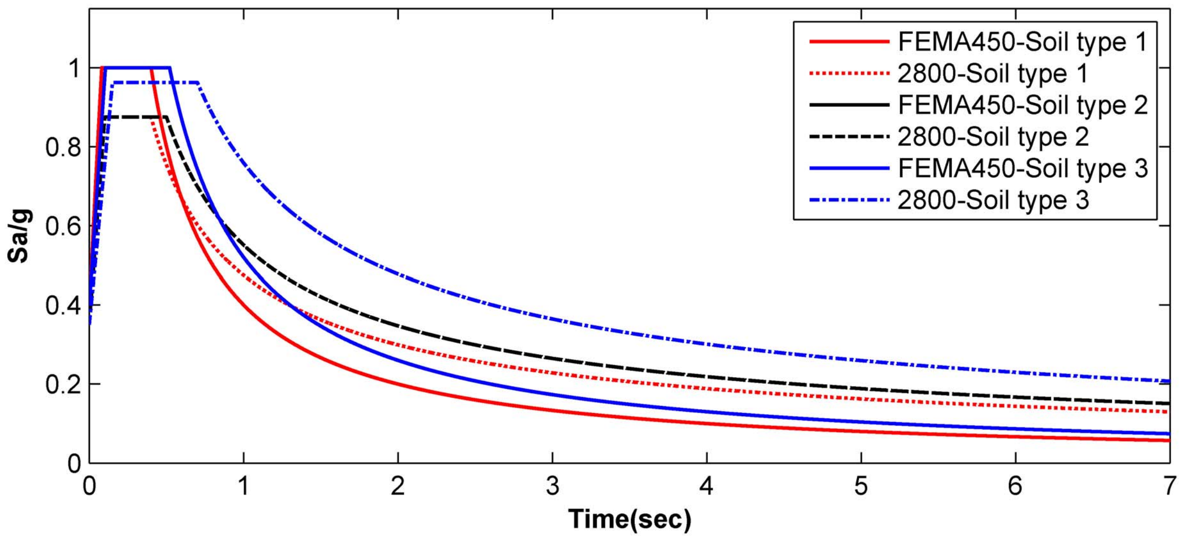

Figure 2. Design spectrum according to FEMA450 and Iranian guidelines.

3.1. FRAMES MODELLING

The seismic nonlinear behaviour of frame elements plays the most important role in the global behaviour of the considered frames. The Opensees framework [11] was utilized for the purpose of response history analysis. The nonlinear beam element with concentrated hinges was employed for the beam modelling. Beams with concentrated plastic hinges and columns of fiber section were employed to simulate the nonlinear flexural behaviour of the moment frames. The beamWithHinges element was chosen for beams. Hence, a pre-determined length at both ends was allocated to plastic hinges and an elastic material was assigned to the mid-span. As the nonlinear behaviour was assumed to be idealized in the hinges, expansion of the nonlinearity to the elastic region was less likely to happen. Therefore, the coefficient of cracking was set equal to 0.5 for the elastic segment of the beams. Nonlinear behaviour of the plastic hinges was defined in accordance with Haselton et al. [12] (Figure 3). Essential relationships are proposed in their study for the calibration of numerous test results in the form of the tri-linear backbone curve suggested by Ibarra ([13], [14]). An important feature of the model is that softening due to concrete crushing, reinforcement buckling, yielding and bond slip can be considered in the negative stiffness region, namely post cap behaviour [12].

The tri-linear Ibarra model, as discussed above, was employed in the Opensees platform using the Clough material proposed by Altoontash [15]. Subsequently, Uniaxial sections with pre-defined according to the Clough material, were assigned to the plastic hinges. It should be noted that all parameters, calculated to form the Ibarra model, were in terms of rotations. In order to make them applicable to a beamWithHinge element, the simple equation (curvature, rotation and plastic hinge length), was used to transform rotations into curvatures which is an advantage of the selected beam element. The plastic hinge's length was set to be equal to beam's height for all cases.

Columns were modelled by means of fiber method with the capability of developing distributed plasticity along the element's length. This choice was made mostly due to the fact that the flexural behaviour in columns is highly dependent on the interaction of their axial and bending forces. However, the aforementioned approach for beams was incapable of considering variable axial forces during the analysis. The fiber sections were assigned to NonlinearBeamColumn elements. Also, each element was divided into four sub-elements in a story level to provide more robustness.

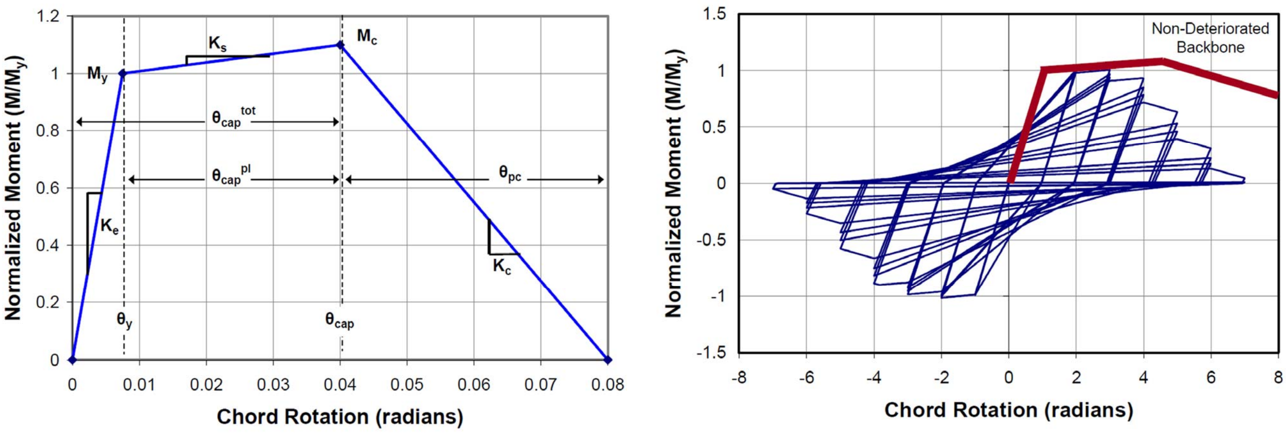

Figure 3. Monotonic and cyclic behaviour of component model [12], [13]

3.2. SHEAR WALL MODELLING

Recently, 'Flexure-Shear Interaction Displacement-Based Beam-Column' element has been developed in the Opensees platform on the basis of the formerly used Multiple Vertical-Line-Element Model (MVLEM). In this new element, the previous multiple vertical columns are defined as fibers of a section. The interaction between the flexural and shear behaviour is provided by assigning a biaxial response to the fibers, incorporating a membrane material model. The flexure-shear interaction displacement-based beam-column element was thus selected to simulate the shear wall element in Opensees platform because of its inclusive features. In particular, with its application, the numerical models showed proper agreement with the characteristics of the designed frames. More information about the element can be found in Orakcal et al. [16].

The definition of the boundary elements was also provided in the model. Hence, the resulting shear wall element would take the form of a single column. However, attaching this column to the moment frame was quite difficult. To cope with this problem, the mid-panel of the shear wall was constructed with the flexure-shear interaction displacement-based beam-column element while the boundary elements were modelled as columns of the main frame. To enhance the robustness, each element was divided into four sub-elements in a story level. End node, located at the same elevation of the boundary elements and the mid-panel column element, was then joined by means of rigid beams. This provided an integrated simulation of the whole shear wall system. As it is obvious, the system could benefit from the top features of both flexure-shear interaction model and fiber section.

3.3. SOIL-FOOTING INTERFACE MODELING

The Beam on Nonlinear Winkler Foundation (BNWF) was employed to model the soil-footing interface[17]. It is capable of simulating the uplift and rocking motions (geometrical nonlinearity) as well as the nonlinear behaviour of the soil (material nonlinearity). Furthermore, it allows for likely changes in soil spring's stiffness and spacing along the foundation length[18]. In this study, the BNWF numerical model was constructed through assigning NonlinearBeamColumn and ZeroLength elements to the strip footing and soil springs, respectively. It is worth mentioning that the beam at the base of the shear wall was set to be rigid due to high flexural stiffness that the shear wall added to the footing's rigidity. In addition, the footing was constrained against sliding ([16]; [19] and [20]). In order to define the Winkler springs, first their properties were determined according to different site classes and the corresponding footing dimensions. Second, Qzsimple1 material (in Opensees) was chosen to represent the soil behaviour according to the computed parameters. Moreover, the Gazetas concentrated stiffness [21] was employed to define the stiffness of the soil springs. Therefore, the distributed stiffness of the Winkler foundation was actually estimated based on continuum approaches. Initially, the total vertical and rotational stiffness of the footing-soil systems were found according to Gazetas proposed relationships. A specific distribution of the Winkler springs with varying stiffness was later selected for each system to produce the same total vertical and rotational concentrated stiffness.

It has been experimentally established

that during the rocking motion, a higher stiffness would develop in the soil medium at the compression zones. The so-called rounding phenomenon happens to retain the stability of the structure [22]. Accordingly, more stiff springs were placed at the ends of the footing strip to supply the rotational stiffness of the soil-footing system. The end lengths were determined based on [22]. Finally, a contribution of vertical springs of particular stiffness, located in the middle and end zones of the footing strip, was chosen based on [23]. Likewise, the strength of the Winkler springs was calculated according to the bearing capacity of the foundations. Among several equations available to determine the bearing capacity, the Terzaghi's relationship (1943) is widely employed in engineering problems [24]. However, a more rigorous form of the Terzaghi's relationship, proposed by Meyerhof (1963) [25], was selected to estimate the foundation bearing capacities in this study.

4. Linear Static Procedure Results

The linear static procedure in ASCE/SEI 41-06 standard is the simplest procedure for the analysis of structures. The lateral loads effects are combined with the gravity loads effects by employing contractive and additive load combinations [3]. However, the contractive load combination was chosen, as written in Equation (1), in this study since it produces more critical situations when the SFSI effects are taken into account [5]. The main reason is that the uplift phenomenon is more likely to happen in the case of contractive load combination when compared with the additive load combination.

$$Q_G = 0.9Q_D \quad (1)$$

where Q_D is the dead load. The lateral base shear is calculated by using Equation (2).

$$V = C_1 C_2 C_m S_a W \quad (2)$$

where C_1 is the modification factor to relate the expected maximum inelastic displacements to the displacements calculated for linear elastic response, C_2 is the modification factor to represent the effect of pinched hysteresis shape, cyclic stiffness degradation, and strength deterioration on maximum displacement response, C_m is the effective mass factor to account for higher mode mass participation effects, S_a is response spectrum acceleration at the fundamental period of structure and damping ratio of the building in the direction under consideration and W is the effective seismic weight of building.

The natural period of the given frames, obtained from eigenvalue analysis, with the corresponding base shear are shown in Table 5.

Table 5. Base shear calculated in the linear static procedure.

No. of Storey	Typer of soil	W (kg)	T (sec)	Effective mass of first mode	V (kg)
3	B	267100	0.124	78.53	296809
	C				315370
	D				347900
6	B	543107	0.369	69.38	619242
	C				645418
	D				687954
10	B	790807	0.790	64.47	411247
	C				546945
	D				646591
15	B	1292088	1.397	61.24	371873
	C				485701
	D				564813

The overturning check is usually done according to Equation (3). On the other hand, if the elements tension strength is taken into account, then, Equation (4) is permitted to be used instead of Equation (3). In Equation (3), M_{OT} is the total overturning moment induced on the element by the seismic forces applied at structural above level under consideration, M_{ST} is the stabilizing moment produced by dead loads acting on the element, R_{OT} is the modification factor to overturning moment. R_{OT} depends on the structure performance in which in the case of IO is taken equal to 4 in the current study.

$$M_{ST} > \frac{M_{OT}}{C_1 C_2 J} \quad (3)$$

$$0.9 M_{ST} > \frac{M_{OT}}{C_1 C_2 R_{OT}} \quad (4)$$

The resulted overturning safety factors are shown in Table 6. The results revealed that the overturning safety factor is less than unity in many cases when Equation (3) is employed. On the other hand, it is always greater than unity in the case of Equation (4).

Table 6. Overturning safety factor in the linear static procedure.

No. of Storey	Type of soil	Equation (3)	Equation (4)
3	B	1.0754	4.3017
	C	1.0121	4.0474
	D	0.9175	3.6700
6	B	0.6449	2.5797
	C	0.6247	2.4990
	D	0.5947	2.3788
10	B	0.6734	2.6937
	C	0.5805	2.3220
	D	0.5246	2.0984
15	B	0.6802	2.7206
	C	0.5478	2.1910
	D	0.4828	1.9313

It is assumed that the sliding phenomenon is not happened in the soil-foundation modelling. To check the validity of this assumption, the sliding strength is calculated according with Equation (5). C is the cohesion of the soil on the surface area of the foundation (A) and f_s is the friction force between the bottom of the foundation and the subsoil.

$$V_{ST} = CA + f_S \quad (5)$$

The sliding safety factor is then determined on the basis of either Equation (6), that is based on R_{OT} factor, or Equation (7), that uses J factor. The resulted sliding safety factors are shown in Table 7 separately based on Equation (6) and Equation (7). The obtained sliding safety factors again show that employing J factor, based on Equation (7) tends to conservative results compared with Equation (6), which employs R_{OT} factor. This result is an important challenge in the ASCE/SEI 41-06 standard which needs to be more elaborated. That is, the issue is more investigated in the following section by using nonlinear response history analyses.

$$SF_{sliding} = \frac{0.9 V_{ST}}{\frac{V_{base}}{C_1 C_2 R_{OT}}} = \frac{0.9(w \tan \phi)}{\frac{V_{base}}{C_1 C_2 R_{OT}}} \quad (6)$$

$$SF_{sliding} = \frac{0.9 V_{ST}}{\frac{V_{base}}{C_1 C_2 J}} = \frac{0.9(w \tan \phi)}{\frac{V_{base}}{C_1 C_2 J}} \quad (7)$$

Table 7. Sliding safety factor in the linear static procedure.

No. of Storey	Type of soil	J	R_{OT}
3	B	0.8954	3.5817
	C	0.7082	2.8327
	D	0.5387	2.1547
6	B	0.9894	3.9578
	C	0.8093	3.2372
	D	0.6511	2.6043
10	B	1.3270	5.3072
	C	1.0407	4.1630
	D	0.8256	3.3024
15	B	1.7923	7.1691
	C	1.2817	5.1267
	D	0.9758	3.9032

5. Nonlinear dynamic procedure results

The nonlinear response history analysis can be accounted as the point of comparison in order to judge which factor (J or R_{OT}) can tend to more realistic results. Nonlinear behaviour of different structural elements were defined as described in Section 3. The gravity load was then applied on each frame before response history begins. The direct integration algorithm was employed in order to perform nonlinear response history analysis.

As the input ground motion selection can significantly change the nonlinear response of structures, the procedure for record selection which has been proposed by Ghafory Ashtiany et al. [26] was employed in this research. The main philosophy in the chosen record selection algorithm is to choose a few strong ground motion records in order to get approximately same result as a large set of records. The computational time is significantly decreased by employing the mentioned method. The records were selected in accordance with the natural period of each frame. The selected records are appropriate to estimate the un-biased median response of structures. The selected records can be found in Table 8 and the corresponding spectra are shown in Figures 4, 5, 6 and 7. Each record is scaled by using Equation (8).

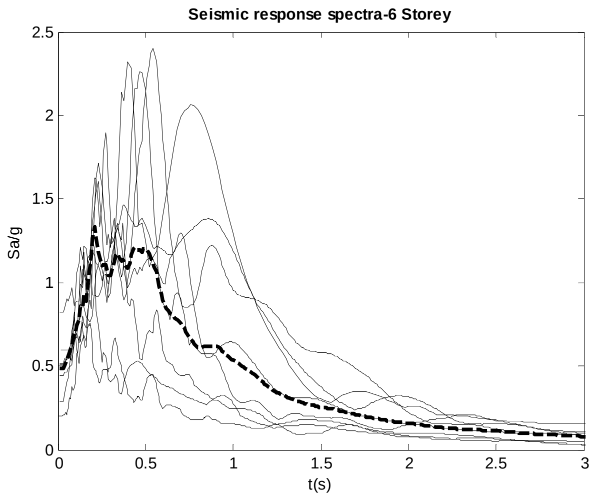

Fig. 4 eight records response spectra for 6 storey frame in the nonlinear dynamic procedure.

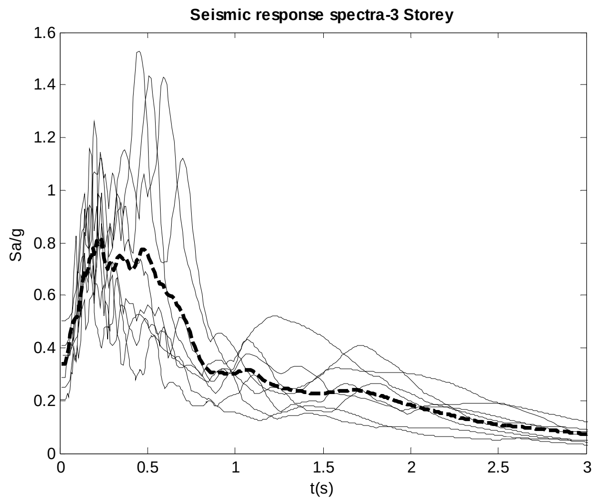

Fig. 5 eight records response spectra for 3 storey frame in the nonlinear dynamic procedure.

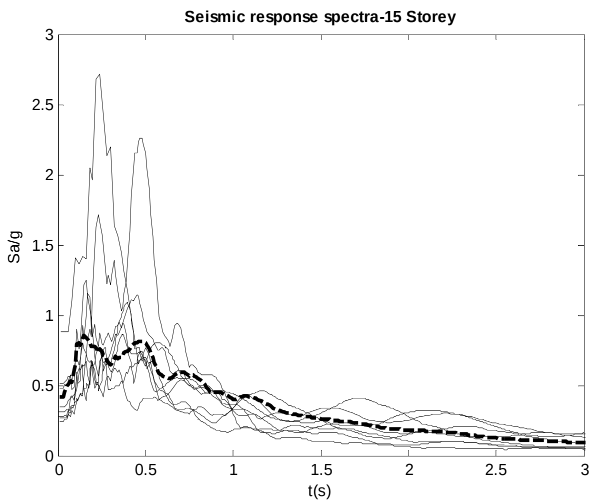

Fig. 6 eight records response spectra for 15- storeyed frame in the nonlinear dynamic procedure.

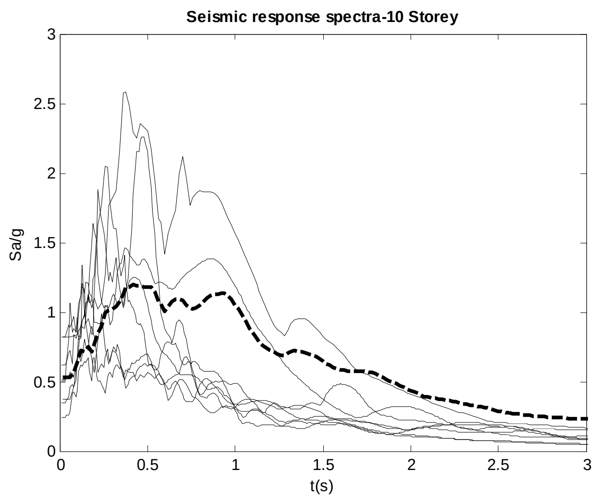

Fig. 7 eight records response spectra for 10 storey frame in the nonlinear dynamic procedure.

$$AccX - Factor = \frac{S_a \times g}{S_{a_{original}}} \tag{8}$$

Table 8. The near-optimal SGMRs for different period ranges [26].

Ground motion subset	Period (sec)	SGM's ID
1	0.1-0.3	3-8-14-20-21-24-27-28
2	0.3-0.5	2-4-10-12-20-21-23-30
3	0.5-0.7	1-4-6-10-12-15-17-23
4	0.7-0.9	1-4-12-22-23-24-25-26
5	0.9-1.25	8-9-12-15-16-22-23-29
6	0.25-2	5-7-13-15-19-23-28-31

where S_a is the response spectrum acceleration at the fundamental period and damping ratio of the building in the direction under consideration, g is the gravity acceleration and $S_{a\ original}$ is the maximum response acceleration which depends on ground acceleration, damping ratio, time step and fundamental period of building.

6. Discussion of the results

Structural analysis using linear and nonlinear procedure usually leads to different results. One of the reasons is modelling nonlinear behaviour of elements and materials in the dynamic analysis. In addition, increasing the energy absorption by the elements in inelastic range, is not considered in the linear procedures. Hence, dynamic procedure result is more convenient and economic in comparison with the static procedure. Thus, to overcome these deficiencies, design codes and standards propose using force reduction factor in linear procedures. According to ASCE/SEI 41-06 standard, J factor has been used in the load combination of force-controlled members (such as foundation) and calculation of overturning and sliding safety factors. Hence, in this section, foundation design forces including base shear and base moment as well as overturning and sliding safety factors were calculated and then compared in the case of linear static and nonlinear dynamic procedure (Figure 8, 9).

As seen in Figure 8 and Figure 9, LSP, Rec1 to Rec8, J and R_{OT} indicate, respectively, to linear static procedure results, result of nonlinear dynamic procedure according to eight selected records and safety factors calculated based on Equation (3), (4), (6) and (7). In the case of non-convergence, e.g. six storeyed frame on the basis of record number two, J factor is set to zero due to the structural global instability.

Comparison of values in Figure 8 and Figure 9 show that there is a large discrepancy between the results. Therefore, in order to reduce the mismatch between the methods, a new value of the force reduction factor (J factor) is calculated. The new J factor based on calculation of overturning and sliding safety factor is determined based on Equation 9.

$$J = \frac{S.F. \times M_{OT}}{M_{ST}} \tag{9}$$

where S.F. is the safety factor calculated based on nonlinear dynamic procedure. The other parameters were defined in the previous sections. Similarly, the new J factor according to calculation of foundation design forces is determined as follows:

$$J = \frac{F_S}{F_D \times C_1 \times C_2} \tag{10}$$

where F_S and F_D are the forces, respectively, in the linear and nonlinear procedures. The calculated force reduction factor based on soil type and number of storeyed for the case of flexible-base assumption is shown in Figure 10. The calculated J factor in this study, as seen in Figure 10, is in accordance with Equation 11.

$$J = \alpha N + \beta \tag{11}$$

where J is referred to new force reduction factor and N referred to number of stories. The coefficients α and β depend on soil type and number of stories as shown in Table 9.

In order to validate the results, J factor based on LSP, in the case of 10 storeyed frame with the flexible base on soft soil condition are shown in Table 10. A good agreement is seen between Table 9 and Table 10 which is an evidence of the applicability of the calculated J factor.

Table10. J factor based on LSP.

moment	**shear**	**sliding**	**overturning**	
2.482	2.161	2.739	4.093	**J**

Table 9. Coefficients to be used in Equation 11.

			No. of Storey		
		soil type	N<6	6≤N<10	10≤N
Overturning	α	B	0.402	-0.011	0.021
		C	0.617	0.052	-0.009
		D	0.613	0.082	0.110
	β	B	0.630	3.137	2.815
		C	-0.116	3.276	3.881
		D	0.087	3.278	2.992
Sliding	α	B	0.402	0.088	-0.066
		C	0.550	-0.106	-0.194
		D	0.542	-0.249	0.191
	β	B	1.105	2.990	4.537
		C	0.483	4.418	5.297
		D	0.481	5.225	0.831
Base shear	α	B	0.268	0.149	-0.041
		C	0.375	0.006	-0.140
		D	0.363	-0.083	0.186
	β	B	0.996	1.712	3.615
		C	0.441	2.655	4.114
		D	0.313	2.988	0.300
Base moment	α	B	0.221	0.139	-0.010
		C	0.235	0.115	0.007
		D	0.258	0.170	0.125
	β	B	0.671	1.160	2.655
		C	0.595	1.314	2.399
		D	0.251	0.782	1.227

7. Conclusion

The gravity and the seismic demand is reduced, in the case of force-controlled members, by J factor on the basis of ASCE/SEI 41-06 standard. The calculated J factor plays a significant role for the final output design. As J factor is limited between 1 and 2, it has been shown in this study that the obtained demand in force-controlled members is relatively conservative. Therefore, a set of 3, 6, 10 and 15-storeyed frames were taken into account. Nonlinear response history analysis was, then, employed in order to evaluate the equivalent linear static procedure. The obtained results show that the base shear as well as the overturning moment in the LSP are always greater than nonlinear dynamic procedure results in which confirms that LSP is a conservative approach. This conservation is more highlighted in the case of taller frames and softer soils. Finally a set of J factors are calculated for different frame height and different soils in order to decrease the conservation in the LSP.

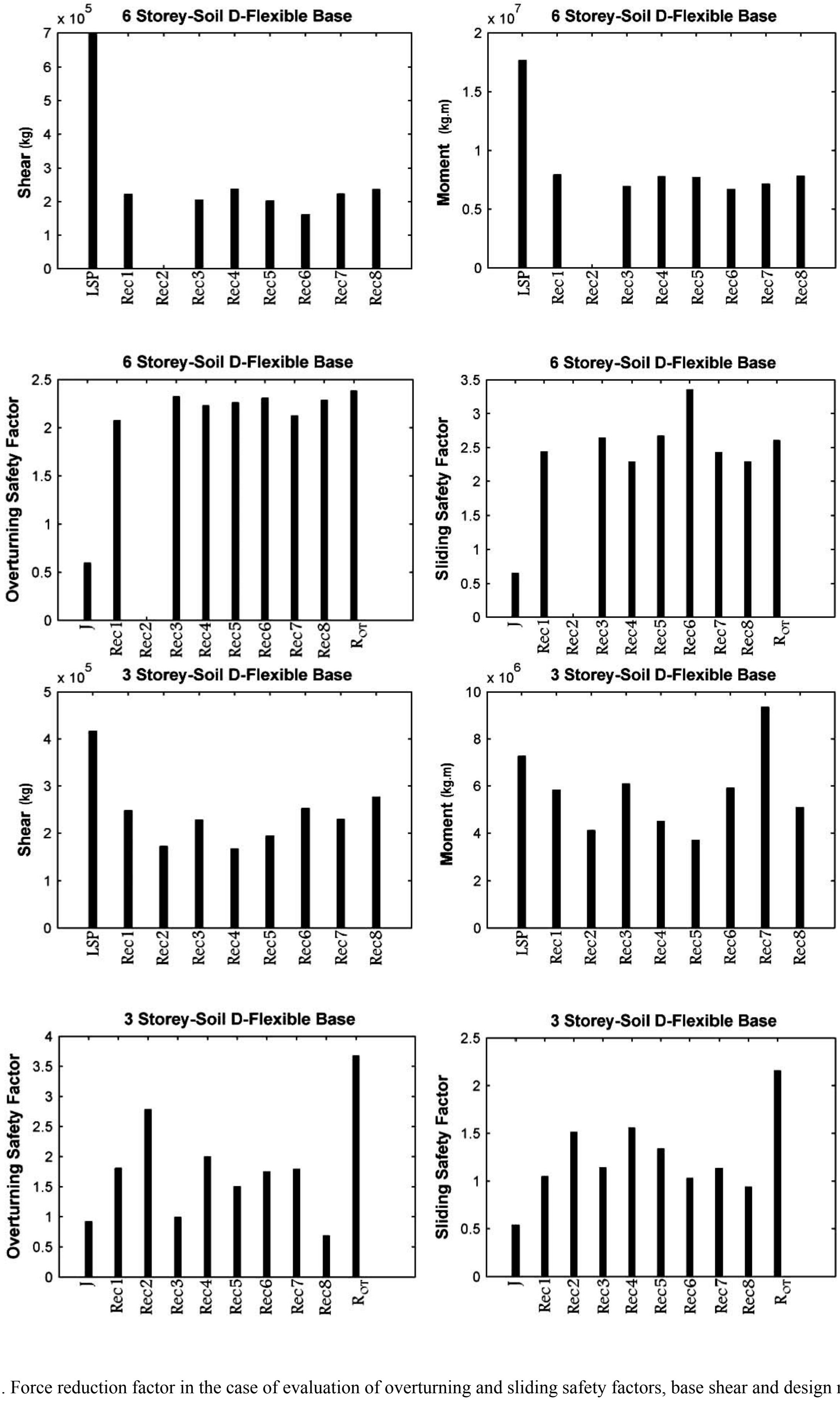

Figure 8. Force reduction factor in the case of evaluation of overturning and sliding safety factors, base shear and design moment of foundations for 3 and 6 storeyed frame.

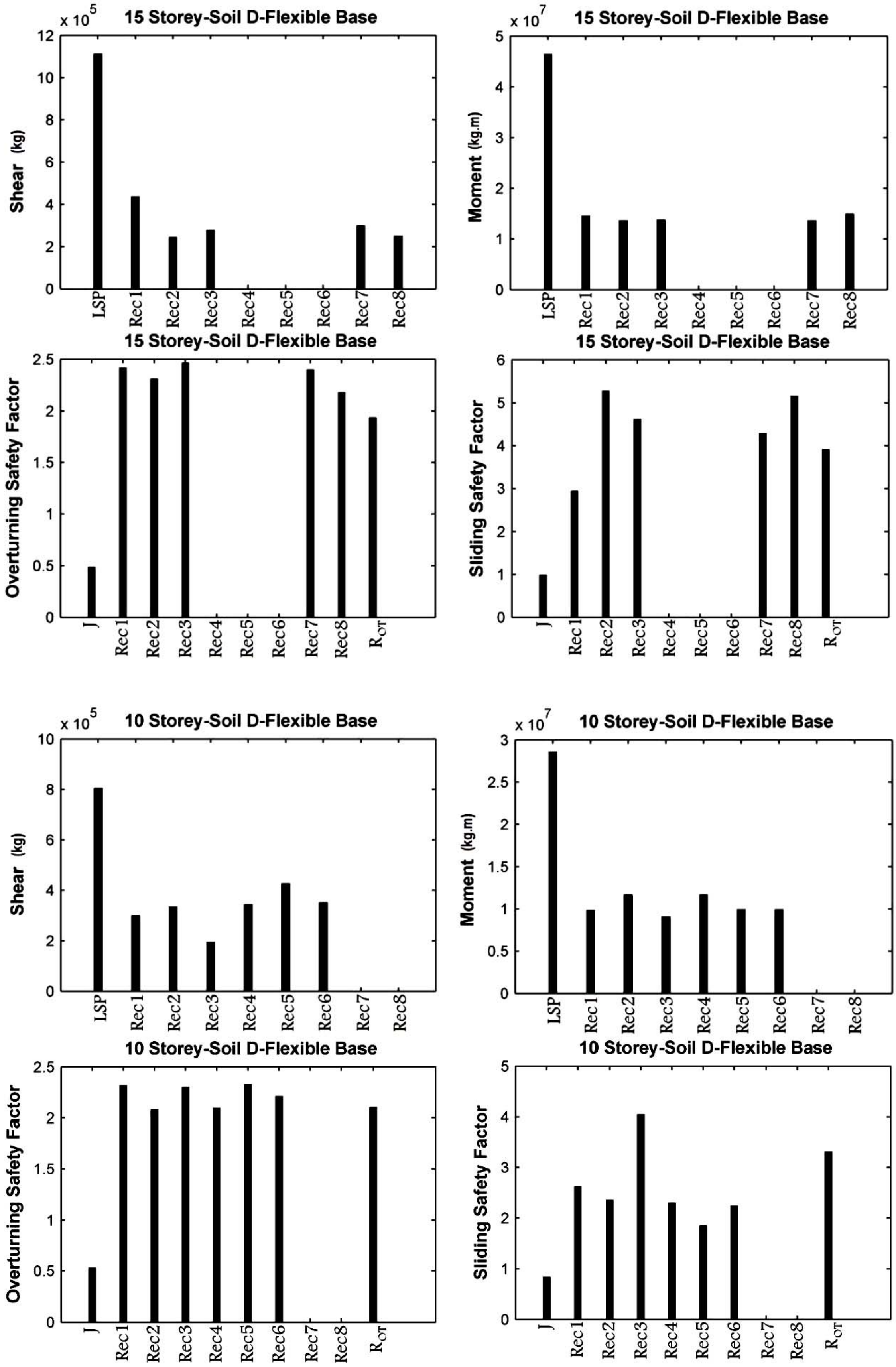

Figure 9. Force reduction factor in the case of evaluation of overturning and sliding safety factors, base shear and design moment of foundations for 10 and 15-storeyed frame.

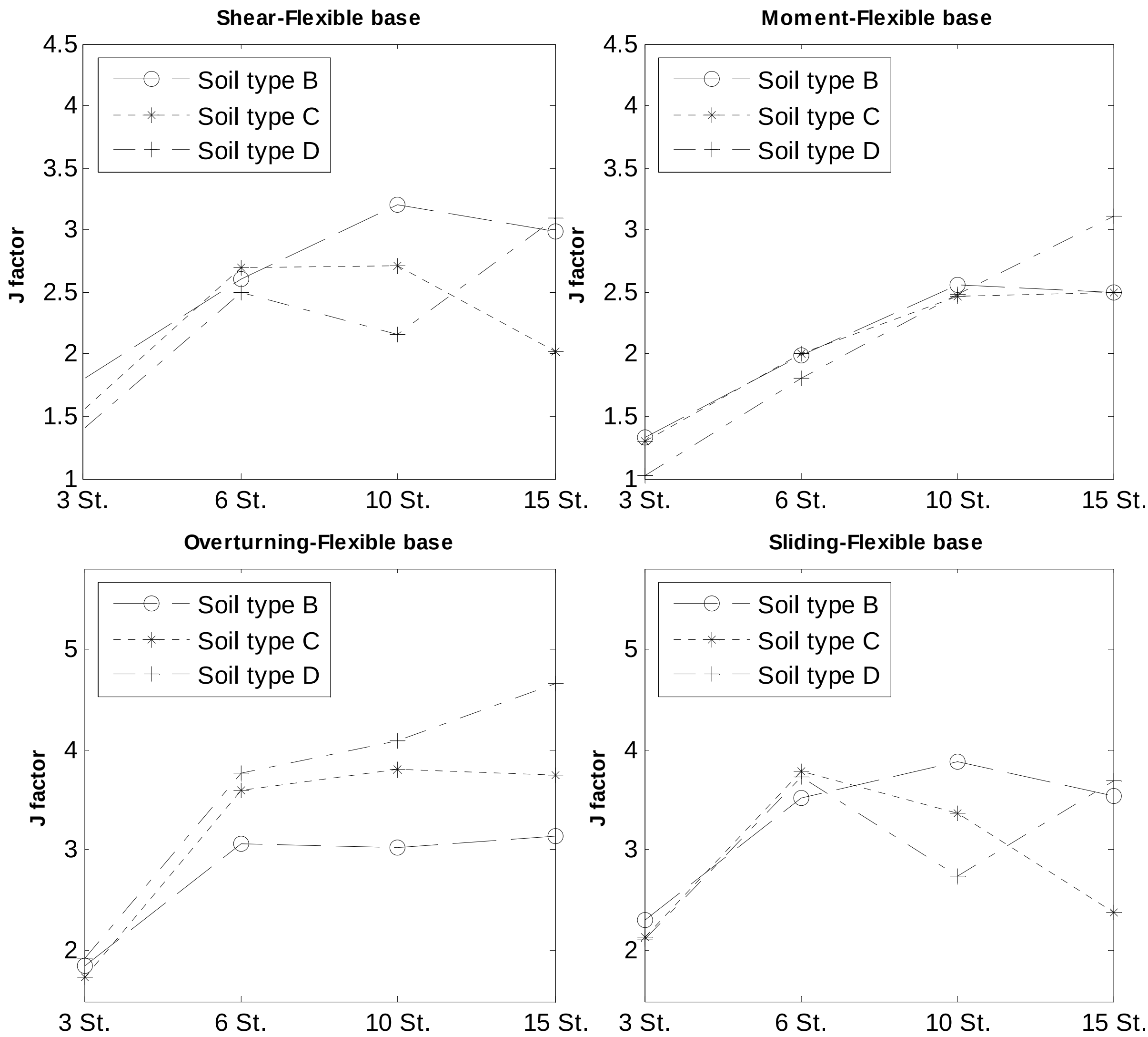

Figure 10. Calculated force reduction factor for the case of flexible-base assumption.

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