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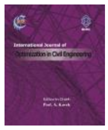

A COMPARATIVE STUDY OF OPTIMUM AND IRANIAN SEISMIC DESIGN FORCE DISTRIBUTIONS FOR STEEL MOMENT RESISTING BUILDINGS

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ABSTRACT

In this study, constant-ductility optimization algorithm under a family of earthquake ground motions is utilized to achieve uniform damage distribution over the height of steel moment resisting frames (SMRFs). SMRF structures with stiffness-degrading hysteric behavior are modeled as single-bay generic frame in which the plastic hinge is confined only at the beam ends and the bottom of the first story columns. Several SMRFs having different fundamental periods and number of stories are optimized such that a uniform story damage (ductility demand) is obtained under a given earthquake ground motion. Then, the optimum lateral load pattern derived from the optimization process is compared with that of the design load pattern proposed by the latest version of the Iranian code of practice, Standard No. 2800 to evaluate the adequacy of the seismic code design pattern. Results of this study indicate that, generally, the average story shear strength profiles corresponding to the optimum seismic design are significantly different from those of the Standard No. 2800 story shear strength pattern. In fact, the height-wise distribution of story ductility demands resulted from utilizing code-based design lateral load pattern are very non-uniform when compared to the corresponding optimum cases. In addition, a significant dependency is found between the average story shear strength pattern and inelastic behavior of structural elements.

Keywords: optimum design; Iranian seismic code; uniform damage distribution; ductility demand; steel moment-resisting frame.

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1. INTRODUCTION

Initial seismic design of regular structures in almost all current seismic design guidelines and codes of practice such as ASCE-7-10 [1], IBC-2015 [2]; Eurocode-8 [3], Iranian Seismic Code, i.e., Standard No. 2800 [4] is based on the equivalent static force (ESF) method. The ESF procedure is fundamentally based on the determination of story shear-strength and stiffness characteristics of the structural systems through utilizing design-compatible spectrum lateral force patterns. It is well known that the code-specified seismic design force patterns, basically established based on the dynamic response of elastic structural systems, do not directly consider the inelastic behavior of the structural system. However, during major earthquake events the expectation is that structural elements will significantly experience different levels of inelasticity. In such cases, code-compliant ESF patterns may not provide an accurate estimation of the earthquake-induced story shear strength and stiffness demands of the structural system. Thus, having selected a proper amount of total stiffness, strength, and also ductile detailing, epically in the plastic hinge regions, in the seismic design process of a project, an engineer may predict and control the global structural damage imparted to the structure. However, an engineer has usually limited control over the height-wise distribution of seismic-induced damage, which is mostly originated from improper distribution of story shear strength and stiffness as well as redistribution influences of inelastic structural responses [5].

In one of the first attempts to find optimum distribution of structural properties, an analytical-based optimization method was developed by Takewaki [6,7] to find stiffness and strength distribution leading to a constant inter-story ductility demand as a structural damage criterion for a shear-building structure subjected to a specified design spectrum. His proposed method is based upon an elastic equivalent linearization technique, and the results revealed that for high-rise buildings it does not lead to a uniform ductility demand (damage) distribution when the structure is subjected to an earthquake induced strong ground motion at the base. Using the concept of energy balance applied to moment-resisting frames with a pre-selected yield mechanism Leelataviwat et al. [8] proposed improved load patterns for non-deteriorating systems. However, the effects of frequency content of ground motions and the degree of nonlinearity were not considered in their suggested load distribution pattern [9].

The comprehensive studies carried out by Goel et al., [10] led to the development of a new seismic design lateral load distribution based on inelastic behaviour of a structure and also a new methodology called Performance-Based Plastic Design (PBPD) for seismic design of a wide ranges of frame systems including moment-resisting frames, eccentrically-braced frames, special truss-moment frames and reinforced concrete frames. In these investigations, performance limit states are pointed out by predictable global yield mechanism and pre-designated target drift limit. The design base shear for each performance level is derived via an energy-based method where the required energy to push the structure up to the target drift is calculated as a fraction of elastic input energy which is obtained from the selected elastic design spectra [10]. By using a similar global yield mechanism, Park and Medina [11] proposed a seismic design methodology for moment-resisting frames to limit the extent of structural damage and distribute this damage uniformly along the height of the non-deteriorating moment-resisting frame structures. They emphasized that designs based on the proposed approach are expected to provide increased protection against global collapse and loss of life during a strong earthquake event. In another study, Hajirasouliha and Moghaddam [12] proposed an effective optimization algorithm for shear-building structures with a remarkably improved convergence speed in order to implement uniform ductility criterion for design of shear buildings. They proposed a new load pattern which was also a function of fundamental period of vibration and target inter-story ductility demand of the structure. Using the same concept, Hajirasouliha and Pilakoutas [13] modified the defined constant coefficients associated with this new pattern to incorporate the influence of site effect without soil-structure interaction phenomenon. The most recent work in this field maybe those of Ganjavi and Hao [14,15], and Ganjavi et al., [16] in which they have investigated the effect of soil-structure systems on the efficiency of different lateral load patterns to achieve the equal ductility demands in all stories of elastic and inelastic soil-structure systems. In one of these researches, Ganjavi and Hao [15] developed a new optimization algorithm for optimum seismic design of elastic shear-building structures with SSI effects. Their adopted optimization method was based on the concept of uniform damage distribution proposed by Hajirasouliha and Moghaddam [12] for fixed-base shear building structures. They proposed a new design lateral load pattern for seismic design of elastic soil-structure systems, which can lead to a more uniform distribution of deformations and up to 40% less structural weight as compared with code-compliant structures. However, their proposed load pattern was developed only for elastic soil-structure systems and, therefore, may not be applicable for non-linear structures. Moreover, their study were based on the results of shear-building structures that may not be applicable for more realistic building structures such as moments-resisting frames that are basically designed based on the “strong- column weak-beam” design philosophy.

In the present paper, steel moment frames structures with different fundamental periods and number of stories are optimized such that a uniform story ductility demand is achieved along the height of the structure subjected to a given earthquake ground motion. Then, the optimum lateral load pattern derived from the optimization process is compared with that of design load pattern proposed by Iranian code of practice, Standard 2800 [4] to evaluate the adequacy of the seismic code design pattern.

2. SELECTION OF GROUND MOTIONS USED IN THIS STUDY

In this investigation, a set of 22 earthquakes ground motions is compiled from five strong earthquakes and utilized for nonlinear dynamic analyses. They were selected from strong ground motion database of the Pacific Earthquake Engineering Center (PEER, http://ngawest2.berkeley.edu/). These earthquake ground motions have been selected based on the following assumptions: (a) They exclude the near-fault ground motion characteristic such as pulse type and forward directivity effects; (b) they are not located on soft soil profiles; hence the effect soil-structure interaction has not been considered in this study (c) They have no long duration characteristics. The selected earthquake ground motions have moment magnitude larger than 6.5 and closest distance to the fault rupture between 14 km and 38 km. These ground motions are recorded on soils 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 [4] . The main specifications of the selected strong ground motions are appeared in Table 1. These ground motions have characteristics consistent with those that dominate the design level seismic hazard (i.e., 10/50) in Iranian code of practice-2800 and the western U.S.

Table 1: Earthquake ground motions used in this study

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

3. GENERIC FRAME STRUCTURAL MODELS

The SMRF models used in this investigation regard to a number of 2-D single-bay, moment-resisting frames with the number of stories N=6, 9, 12 and15with fundamental periods computed by $T= 0.15N$ as shown in Fig. 1. Using this type of generic frame to analyze the seismic response of multi-bay building frames has been employed by some researchers [5,11,17-19]. This approach has attracted researchers for seismic performance assessment since it represents a less computational effort for performing repeated nonlinear dynamic time history analyses. Results obtained by the researchers demonstrated that single-bay generic frame models are adequate to represent the global dynamic behavior of more complex regular multi-story frames exposed to earthquake excitations [5, 11]. The main properties of the generic frames used in this paper are: (1) Models are one- bay two-dimensional steel moment-resisting frames. (2) The distribution of story mass is uniform over the floor levels. For all SMRFs, story height is constant and equal to 3.6 m. Moreover, the beam span is equal to 7 m. (3) The effect of finite joint regions is not taken into account, meaning dimensions of centerline are considered for column and beam members. (4) The generic SMRFs are designed based on the strong column-weak-beam philosophy. In other words, the plastic hinge is confined only at the beam ends and at the bottom of the first story columns as shown in Fig. 1. (5) When the frame is undergone to a given lateral load pattern, the same value of overstrength is supposed at all stories, which means that beams and columns strengths are adjusted such that yielding occurs simultaneously at all plastic hinge locations. This provides the computation of inter-story ductility ratio which in its turn is obtained from yield story drift. (6) The first mode shape for all the models is a straight-line, which regards to the fact that each story stiffness is adjusted such that as the frame is under a triangular load pattern, a uniform height-wise distribution of story drifts over the height is occurred. In this manner, the relative height-wise distribution of member stiffness is also achieved. (7) In time history dynamic analysis, structural damping is modelled based on Rayleigh damping model with 5% of critical damping assigned to the first mode as well as to the mode where the cumulative mass participation is at least 95%. (8) The moment-rotation hysteretic behavior is modeled by using rotational springs with Modified Clough Bilinear stiffness-degrading model with 3% strain hardening as depicted in Fig. 2. (9) Member P-Delta is not taken into account for, whereas the P-Delta for the whole structure which is called as global effect is considered through quantifying the elastic first story stability coefficient as proposed by Medina and Krawinkler [19].

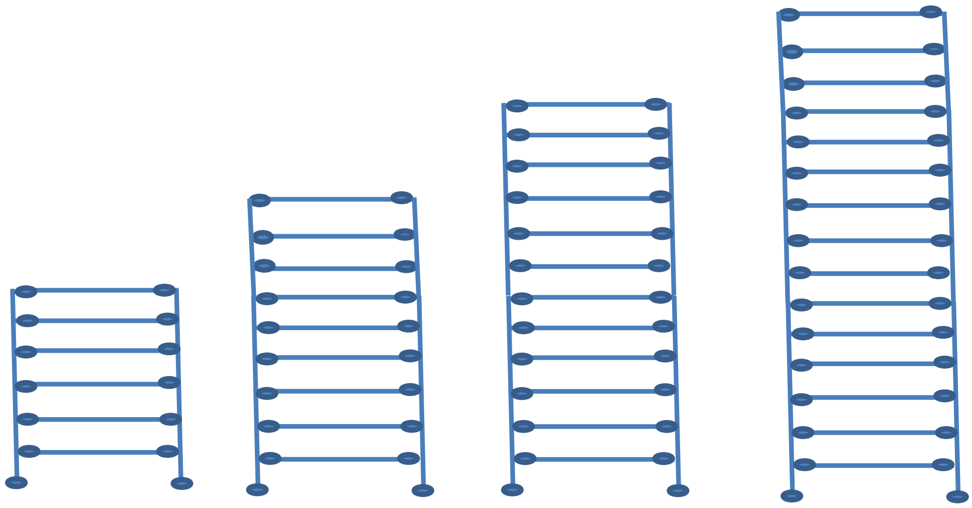


Figure 1. Single-bay steel moment-resisting frames with *N*= 6, 9, 12 and 15

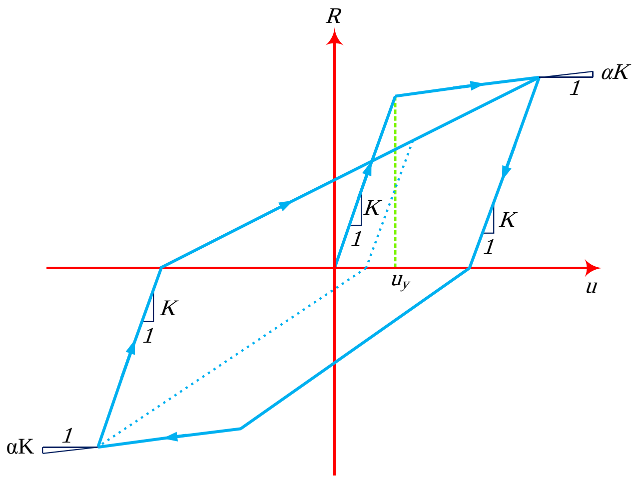


Figure 2. Modified Clough Bilinear Stiffness-Degrading Model

4. VERTICAL DISTRIBUTION OF SEISMIC LATERAL FORCES BASED ON STANDARD-2800

The suggested formula for the lateral load pattern specified by Iranian Seismic Code of practice i.e., Standard- 2800 [4] is defined as:

$$F_x = C_{vx} . V_b \tag{1}$$

$$C_{vx} = \frac{w_x H_x^k}{\sum_{j=1}^{n} w_z H_z^k} \tag{2}$$

where F_x and C_{vx} is relative story shear strength and shear strength coefficient at level x, respectively. V_b is defined as the sum of the absolute design lateral force which is exerted at the base level of the building. The portion of the total gravitational force of the structure at the level x or z are denoted by w_x and w_z that is equal to the dead plus a percentage of live load at this level; the height from the base to the level x or z is defined by H_x and H_z. I is obvious that in equivalent static procedure which is based on the elastic analysis and only the first mode of vibration is taken into account. Therefore, in order to incorporate the higher mode effect, the new parameter k which, defined as an exponent related to the effective fundamental period of the structure, is introduced. As described in Standard No. 2800, for structures with a fundamental period of vibration T equal or less than 0.5 s the value of k is equal to 1, and for T equals to 2.5 s or larger, the k value must be equal to 2.0. And for structures having a fundamental period of vibration between 0.5 and 2.5 s, k can be calculated by linear interpolation between 1 and 2. It should be noted that when the k value is equal to unity, the relative story shear strength pattern conforms to an inverted triangular lateral force distribution over the height; whereas, when k is equal to the value of 2.0, the lateral load profile conforms to a parabolic lateral force pattern with its vertex at the base level. The relative shear force distribution described in Eq. 2 is based on the assumption that the first mode is a straight line. In addition, when k is equals to 1 and 2, the structural responses are supposed to be controlled mainly by the first mode and higher mode effects, respectively. Note that the shape of the above load profile is only affected by the fundamental period of vibration of the structural system T, as well as the height-wise distribution of the mass and stiffness, whereas the influence of the level of inelastic behavior is not accounted for in the distribution of lateral forces over the height. The latter point is very important because when structures are subjected to severe ground excitations, some structural elements may be prone to yielding, and consequently experience significant levels of inelastic behavior. Hence, direct consideration of the inelastic behaviour in the seismic design lateral load pattern of structure seems to be necessary as reported by several researchers [5,11, 14, 16,18, 19].

5. OPTIMIZATION ALGORITHM TO ACHIEVE OPTIMUM LATERAL LOAD PATTERNS FOR SMRF STRUCTURES

As mentioned in the literature, the main objective of this study is distributing structural damage along the height of a moment-resisting frame which is regarded as "*Optimum Design*". The required relative shear strength pattern corresponding to this performance target is called optimum lateral load pattern which can be compared with the design lateral load pattern proposed by Iranian seismic code of practice, Standard No.2800 [4]. In such a case, one can easily evaluate the efficiency of the code-based design lateral load pattern for SMRF structures when subjected to a family of realistic earthquake ground motion excitations. In this regard, it is mandatory to select proper engineering response or demand parameters to determine the distribution of damage of the structure. Among them, inter-story and global ductility ratios, maximum inter-story drift ratio, the number of cycles of yielding, cyclic story ductility, normalized hysteretic energy and also a combination of above-mentioned parameters are those of such engineering demand parameters that are commonly used by researches to compute seismic damage imparted to a structure [5, 11, 18, 19]. Two of the aforementioned parameters are widely used by many researchers to quantify the structural damage for non-deteriorating structural systems: (1) maximum inter-story drift ratio which is defined as the maximum relative displacement between two consecutive story levels normalized by the story height and (2) inter-story ductility ratio which is defined as the maximum inter-story drift normalized by the inter-story yield drift.

Generally, a steel structure with ductile structural elements with no strength deterioration can withstand forces and carry larger loading without losing its carrying capacity entirely. In performance based-seismic design, the maximum story drift and ductility ratios are usually two of the most appropriate parameters to determine the structural damage. It is believed that they have several advantages such as (a) they are very simple parameters to be computed by researchers; (b) They are perceptible for all structural engineers; (c) many experimental studies have been carried on these parameters. Therefore, They can be considered as sufficient earthquake engineering demand parameters to evaluate the structural damage imparted to the building structures during an earthquake event. In this study, these parameters are selected as suitable indicators of structural damage. The following step-by-step iteration process is proposed for the generic SMRF buildings under a given earthquake ground motion to achieve optimum relative shear strength pattern along the height of the structure:

1. Define a generic SMRF model with specific number of stories, i.e., N=?
2. Select the target fundamental period (T_1). Calculate and assign member stiffness based on the first mode shape of shear-type structure through pushover analysis. An iteration process should be conducted to achieve a presumed fundamental period of vibration.
3. Select the target story ductility ratio, μ_t=?
4. Perform nonlinear pushover analysis and assign member strengths based on an arbitrary seismic design lateral force pattern such as code-based pattern.
5. Select an earthquake ground motion and scale it based on Standard No.2800 [4] for the 10/50 ground motion hazard level, which is defined as that corresponding to 10 percent probability of exceedance of a given ground motion intensity measure in 50 years.
6. Perform nonlinear dynamics time history analysis and calculate the maximum inter-story ductility ratio, $\mu_{max(i).}$ Control the following condition:

$$if \quad \gamma_i = \left| \frac{\mu_{\max i -} \mu_t}{\mu_t} \right| \times 100 \le 0.5 \;\rightarrow\; \text{the patten is "Optimum"} \tag{3}$$

If this is the case, the story shear strength at each story must be modified by a correction factor of $(\mu_{\max i}/\mu_t)^{0.05}$. The process of updating the height-wise distribution of story shear strength is repeated until γ_i are less than 0.5.

7. The steps 1 to 6 are repeated for other models having different number of stories, fundamental periods, ductility ratios and earthquake ground motions.
8. Now the obtained lateral load patters which is regarded as "*Optimum Pattern*" can be compared with the one proposed by Iranian code of practice, Standard No. 2800 [4].

6. EFFICIENCY OF THE PROPOSED OPTIMUM SEISMIC DESIGN PROCEDURE

To examine the efficiency of the proposed method for optimum seismic design of non-linear steel moment-resisting frame structure, the algorithm described in the previous section is utilized to a 12-story building with T= 1.8 sec, and μ_t= 2 and 6 representing low and high levels of inelasticity subjected to the selected earthquake ground motions shown in Table 1. Fig. 3 illustrates a comparison of the average results obtained from Standard No.2800 design lateral load pattern [4] with those of the proposed optimum load pattern. It is shown that the optimum design lateral load patterns, in general, can be very different from code-based lateral load patterns. On the other hand, a significant difference is observed between the ductility demand profiles resulted from applying these two load patterns . In fact, the height-wise distributions of story ductility demands resulted from utilizing code-based design lateral load pattern are very non-uniform with respect to the corresponding optimum cases. As a well-known index for indicating the efficiency of the load pattern, the Coefficient of Variation (COV) of story ductility demands resulted from applying Standard No. 2800 [4], and optimum patterns are calculated, which are 47%, 1.2% for μ_t= 2, and 58% and 1.8% for μ_t= 6, respectively. This implies that utilizing code-based load pattern cannot result in an optimum seismic performance of SMRF structures in inelastic range of vibration. In fact. structural inelastic behaviour has not been properly taken into account for Iranian seismic code of practice load pattern.

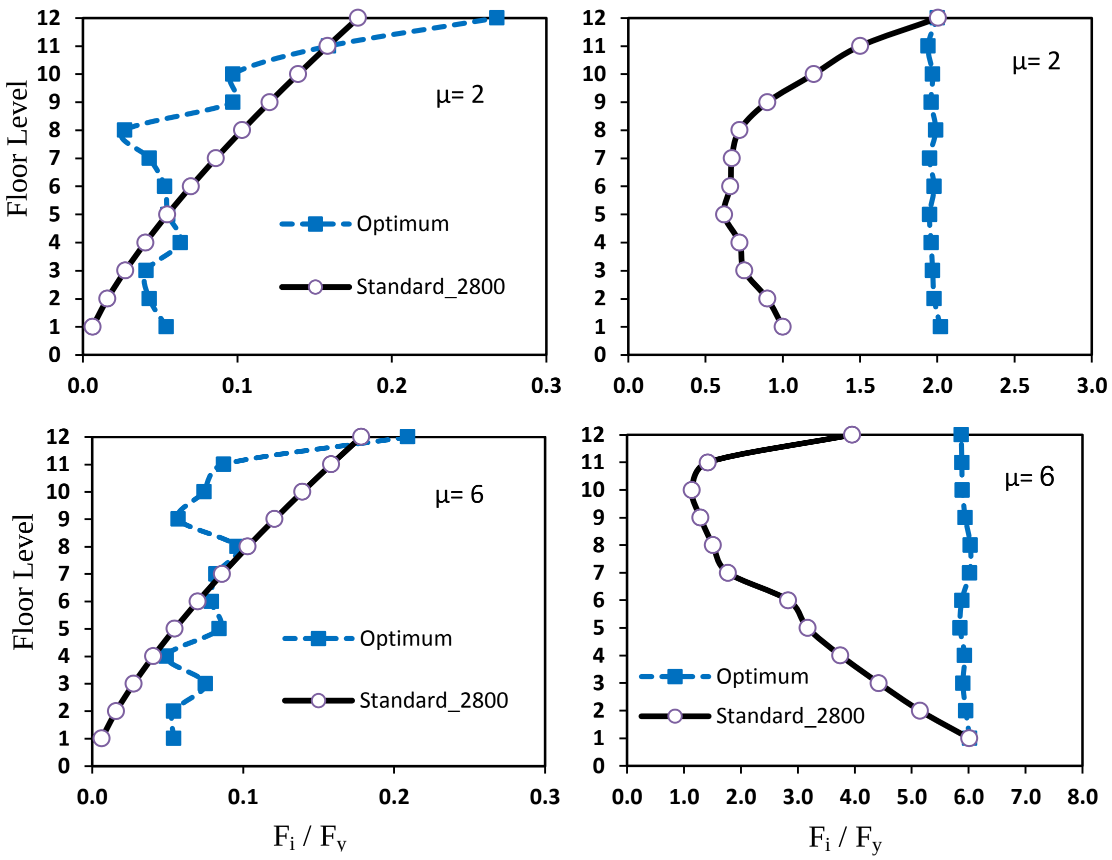
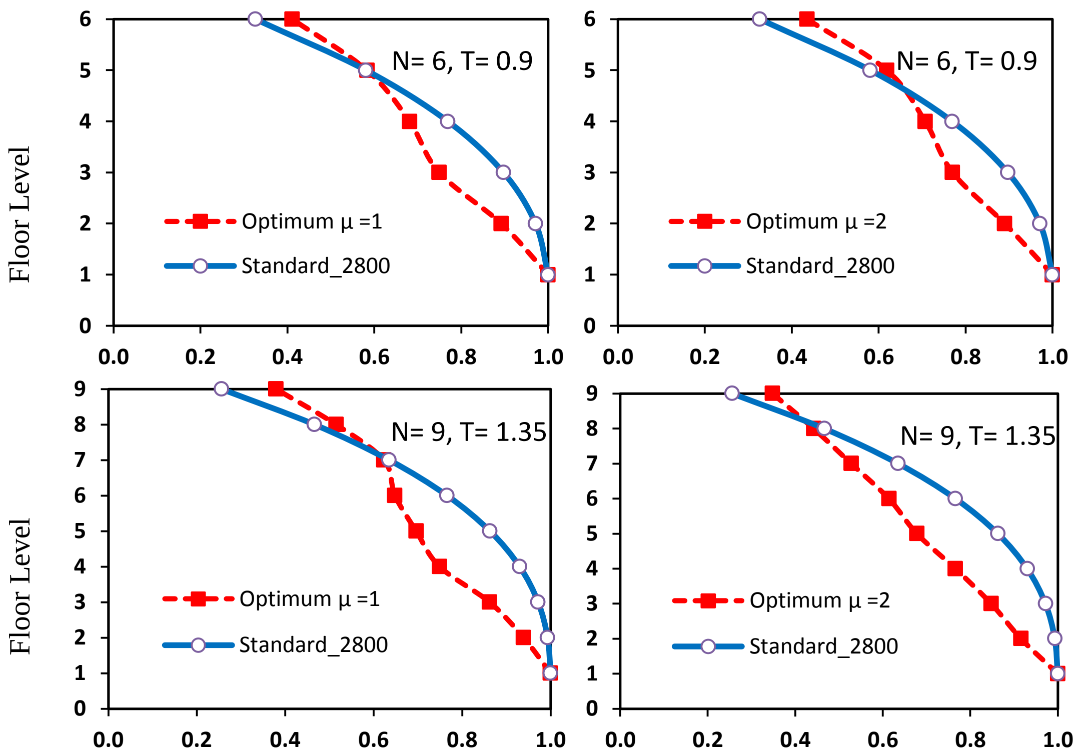


Figure 3. Comparison of mean lateral load and story ductility profiles, SMRF with N = 12, T = 1.8 s , for (a) μ_t= 2 , (b) μ_t= 6

7. COMPARISON OF IRANIAN DESIGN SHEAR STRENGTH PATTERN WITH OPTIMUM PATTERN

As shown in the previous section, the steel moment resisting frame structures designed based on the code-specified lateral force patterns and exposed to strong ground motions exhibit a non-uniform distribution of damage (i.e., ductility ratio) along the height. In this study, it is assumed that inter-story ductility demands are an adequate measure of structural damage. Thus, in performance-based seismic design, besides controlling the amount of maximum inter-story ductility demand as an index of structural damage, designing structures leading to a uniform distribution of structural damage along the height is also an desirable performance target which is called optimum design. Here, to examine more comprehensively the adequacy of the design lateral load pattern of Standard N. 2800, a parametric study has been carried out by utilizing the proposed iterative optimization algorithm described in the previous section to estimate the optimum shear strength patterns for each of 22 ground motions and structural models. It is emphasized that the optimum shear strength pattern corresponds to the required story shear strength profile to achieve a uniform distribution of inter-story ductility demands along the height. Figs. 4 and 5 show the average story shear strength patterns as a function of the target story ductility ratio of 1, 2, 4 and 6 for all the SMRFs with N= 6, 9 and 15. The design shear strength pattern based on Iranian seismic code provision of Standard No. 2800 (Eq. 2) is plotted in all graphs for comparison. As can be seen in Eq. 2, the inelastic behavior (ductility demand) has not been taken into account for in shear strength distribution over the height. Generally, the average story shear strength profiles corresponding to the optimum seismic design (i.e., a uniform target story ductility distribution over the height) are drastically different from those of the Standard No. 2800 pattern. From these figures the following conclusion can be drawn: (1) a significant dependency can be found between the average story shear strength pattern and inelastic behavior through the target inter-story ductility. (2) for a structures with a specific number of stories, as the ductility demands increases, the required strength demands in top stories decreases which is more pronounced for the taller structures. This behavior is could be due to the effect of higher modes, which is more prominent when the structure experiences lower level of inelastic behavior. This conclusion is consistent with those reported by Ganjavi and Hao [14,15] for soil-shear building structures, Medina and Krawinkler [19], and Park and Medina [11]for regular steel moment frames.



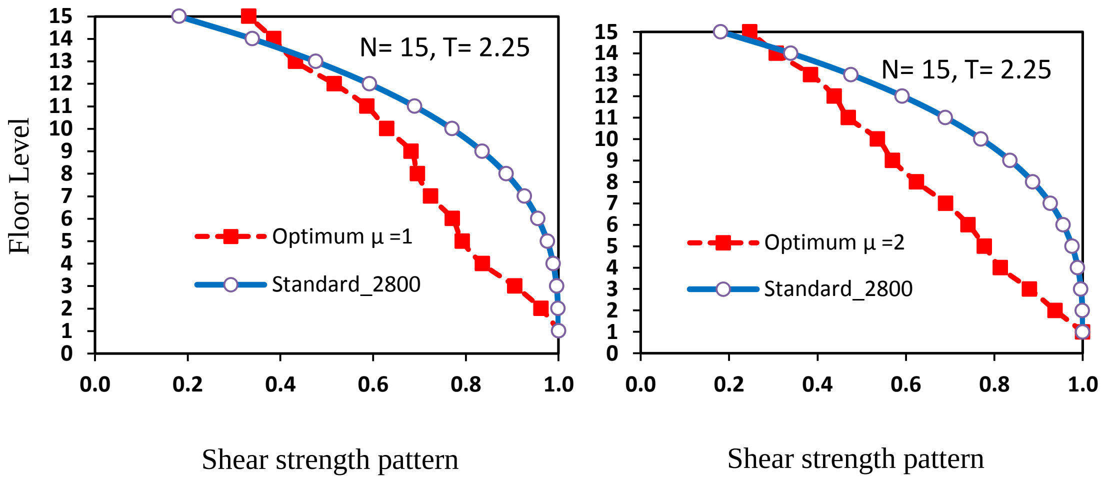
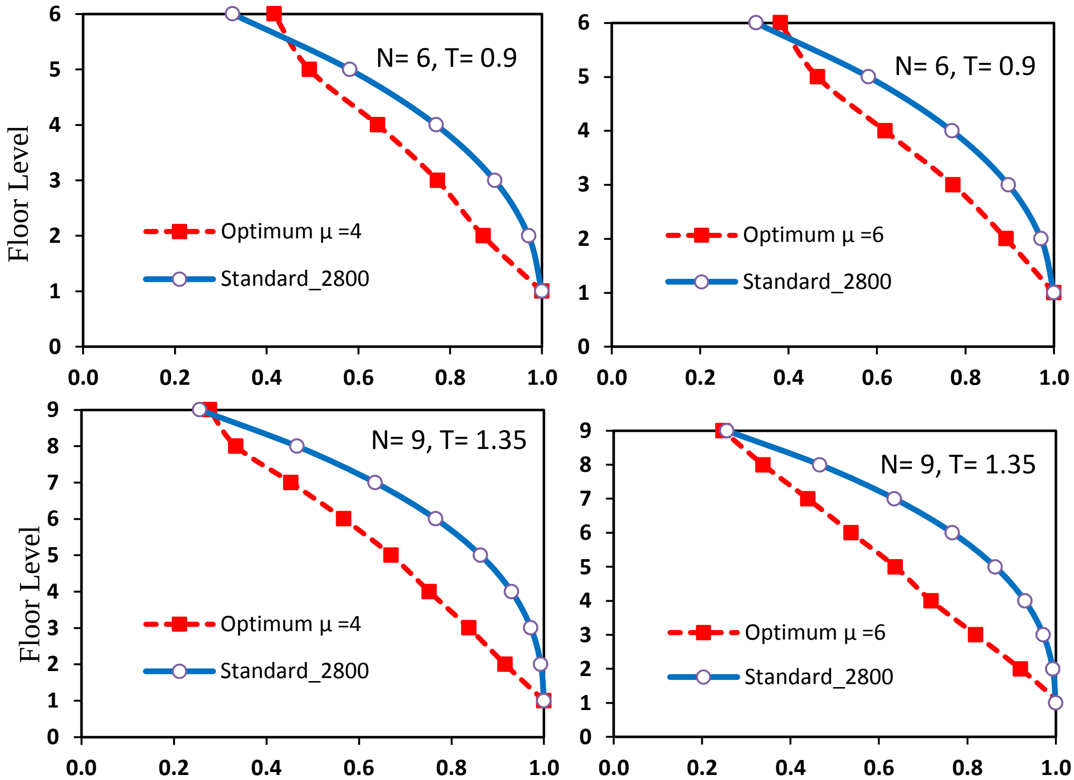


Figure 4. Comparison of average of Optimum and Standard No. 2800 shear-strength profiles for SMRFs with N = 6, 9, 15 with target ductility ratios of 1 and 2 (Average of 22 ground motions)



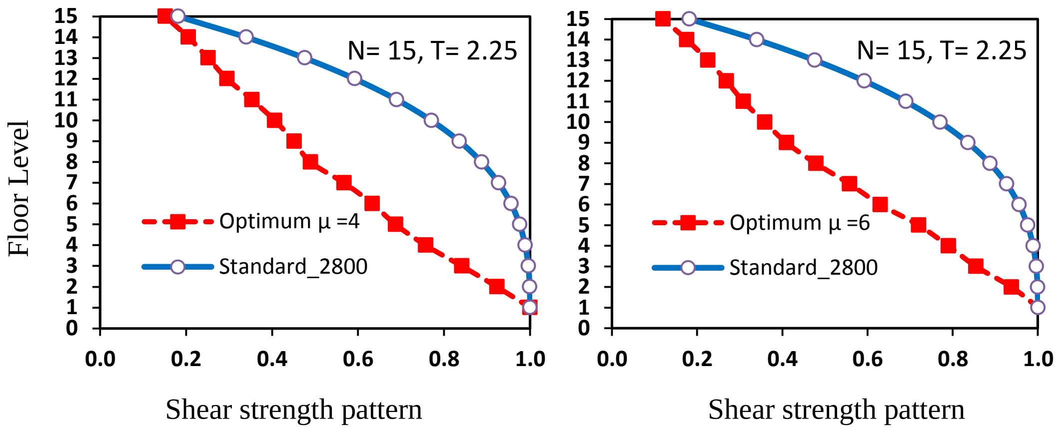


Figure 5. Comparison of average of Optimum and Standard No. 2800 shear-strength profiles for SMRFs with N = 6, 9, 15 with target ductility ratios of 4 and 6 (Average of 22 ground motions)

The Coefficient of Variation (COV) of story ductility demands resulted from applying Standard No. 2800 [4], and optimum patterns are calculated for all target ductility ratios, and the average results are shown in Fig. 6. As seen, the height-wise distributions of story ductility demands resulted from utilizing code-based design lateral load pattern are very non-uniform when compared to the corresponding optimum cases in all levels of inelastic behaviour.

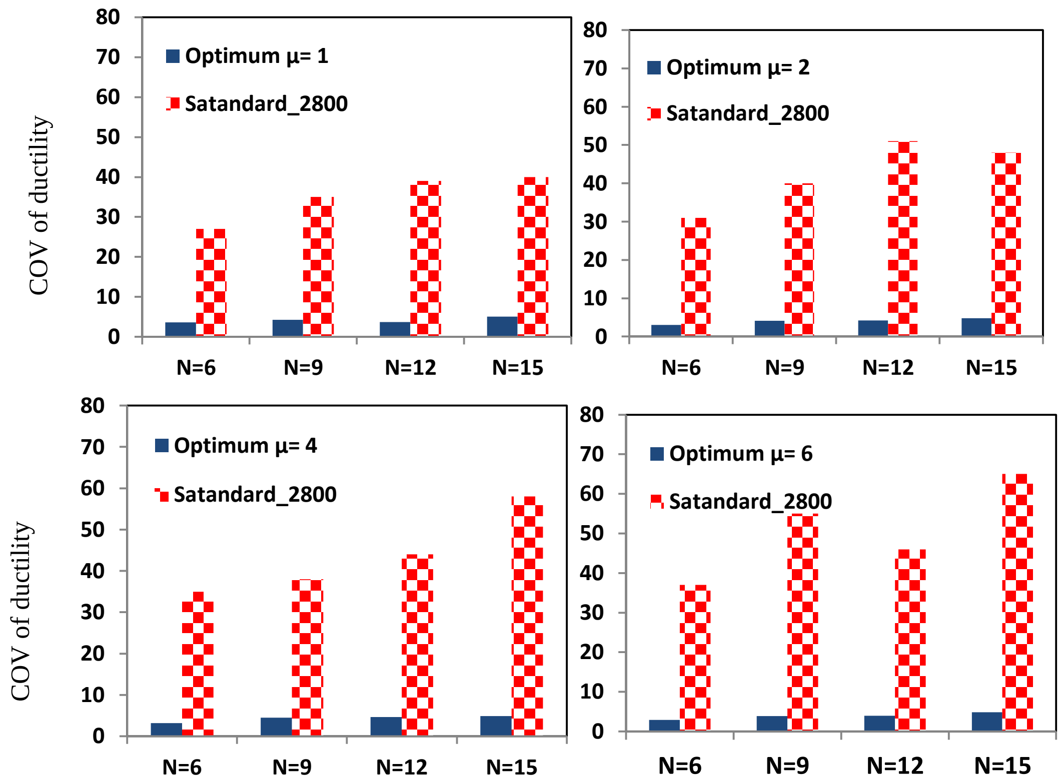


Figure 6. Average of COV of ductility demand distributions for SMRFs with N = 6, 9, 12, 15 with target ductility ratios of 1, 2, 4 and 6 (Average of 22 ground motions)

8. CONCLUSION

In the present paper, steel moment frames structures with stiffness-degrading hysteric behavior having different fundamental periods and number of stories are optimized such that a uniform story damage (ductility demand) is achieved along the height of the structure subjected to a given earthquake ground motion. Then, to evaluate the adequacy of the seismic code design pattern, the optimum shear strength pattern derived from the optimization process is compared with the design load pattern proposed by Iranian code of practice, Standard No. 2800 [4]. Results of this study indicate that, generally, the average story shear strength profiles corresponding to the optimum seismic design (i.e., a uniform target story ductility distribution over the height) are significantly different from those of the Standard No. 2800 ones. In fact, the height-wise distribution of story ductility demands resulted from utilizing code-based design lateral load pattern are very non-uniform with respect to the corresponding optimum cases. In addition, a significant dependency was found between the average story shear strength pattern and inelastic behavior through the parameter of target inter-story ductility ratio. It is also demonstrated that as the ductility demands increases, the required strength demands in top stories decreases which is more pronounced for the taller structures. From a technical point of view, since controlling the maximum value and height-wise distribution of story ductility demands can have significant role on P-Delta sensitive structures, more investigation should be performed to modify the code-based seismic load pattern to take into account for inelastic behavior.

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