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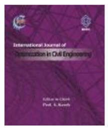

ASSESSMENT OF DUCTILITY REDUCTION FACTOR FOR OPTIMUM SEISMIC DESIGNED STEEL MOMENT-RESISTING FRAMES

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ABSTRACT

In the present study, ten steel-moment resisting frames (SMRFs) having different numbers of stories ranging from 3 to 20 stories and fundamental periods of vibration ranging from 0.3 to 3.0 second were optimized subjected to a set of earthquake ground motions using the concept of uniform damage distribution along the height of the structures. Based on the step-by-step optimization algorithm developed for uniform damage distribution, ductility-dependent strength reduction factor spectra were computed subjected to a given far-fault earthquake ground motion. Then, the mean ductility reduction factors subjected to 20 strong ground motions were computed and compared with those designed based on load pattern of ASCE-7-16 (similar to standard No. 2800) code provision. Results obtained from parametric studies indicate that, except in short-period structures, for moderate and high levels of inelastic demand the structures designed based on optimum load pattern with uniform damage distribution along the height require larger seismic design base shear strength when compared to the frames designed based on the code provisions, which is more pronounced for long-period structures i.e., the structural system becomes more flexible. This phenomenon can be associated to the P-delta effect tending to increase the story drift ratios of flexible structures, especially at the bottom stories. For practical purpose, a simplified expression which is a function of fundamental period and ductility demand to estimate ductility-dependent strength reduction factors of designed SMRFs according to code-based lateral load pattern is proposed.

Keywords: optimum design; generic frame; ductility reduction factor; uniform damage distribution; steel moment-resisting frame; practical equation

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1. INTRODUCTION

In strong earthquake ground motions, the design base shear strength recommended in seismic provisions is typically much lower than the base shear strength that is required to sustain the structure in the elastic range. In the force-based seismic design method, a design lateral force for a given structure is computed based on an elastic design acceleration response spectrum, which is called the design base shear. To consider the inelastic behavior, the design shear force of a given structural system obtained from the elastic acceleration response spectrum is reduced by a strength reduction factor which is strongly dependent on the energy dissipation capacity of the structural systems. The structure is then designed for the reduced shear strength, and the displacement or inter-story drift can be controlled so that the code-compliant limits are coped with. Strength reductions from the elastic strength demand are commonly accounted for through the use of strength reduction factor, which is one of the most controversial issues in the seismic-resistant design provisions. The code-specified values of strength reduction factors in different seismic provisions even for the same type of structure are usually different, reflecting the fact that the recommended values could be to a large extent based on judgments, experiences and observed behavior of structures during past earthquake events instead of analytical results.

For a single-degree-of-freedom (SDF) system, strength reduction factor refers to the seismic force at the predefined design level and can be considered as a product of the conventional ductility reduction factor, reflecting the nonlinear hysteric behavior in a structure, and the over-strength reduction factor that account for other reduction factors such as reductions due to element over-strength, redundancy, strain hardening and etc. During the past forty years, extensive studies have been conducted on ductility reduction factor (DRF). The pioneering investigations performed by Veletsos and Newmark [1] and Newmark and Hall [2] may be regarded as the first renowned studies on DRF. Based on elastic and inelastic response spectra of NS component of El Centro earthquake as well as previous studies on SDOF systems to pulse-type excitations, Newmark and Hall [2] proposed simplified expressions for DRF as a function of target period and ductility ratio of the structure. In another study, based on mean inelastic spectra of 20 artificial ground motions compatible with the Newmark-Hall elastic design spectra, Lai and Biggs in 1980 [3] proposed alternative expressions as a function of target ductility, period as well as period ranges. Many more studies were made by researchers to propose simplified equations for strength reduction factor of SDOF systems including Elghadamsi and Mohraz [4], Fischinger et al., [5], Miranda and Bertero [6], Lam et al., [7], and Karmakar and Gupta [9].

These studies were mainly based on the dynamic response of SDOF systems while real structures are multiple degrees-of-freedom (MDF) and complex behaviors such as contributions to structural responses from higher modes cannot be captured with the SDF systems especially in the inelastic response range. A parametric study was conducted by Nassar and Krawinkler [9] on three types of simplified MDF models to estimate the modifications required to the inelastic strength demands obtained from bilinear SDOF systems in order to limit the story ductility demand in the first story of the MDOF systems to a predefined value. More examples of the works conducted on the subject can be found in the references [10-12]. It is known that structural configuration in terms of stiffness and strength distributions can affect the seismic response and behavior of structures. During the past decade, several researchers have investigated the effect of strength and stiffness distribution on strength and ductility demands of building structures [13-18]. Ganjavi and Hao [19] through conducting an intensive parametric study investigated the effect of structural characteristics distribution including story shear strength and stiffness patterns on strength demand and ductility reduction factor of shear-building fixed-base and soil-structure systems subject to a large number of earthquake ground motions. They concluded that for both fixed-base and flexible-base models, with exception of those with very short periods, the lateral sorry strength and stiffness patterns can significantly affect the total strength demands. They also found that using the results of the uniform story strength and stiffness distribution pattern which has been the assumption of many previous research works would result in a significant overestimation of the strength demands, generally from 2 to 4 times, for shear-building systems designed in accordance with the code-compliant design patterns.

In the present study, 10 steel-moment resisting frames (SMRFs) having different numbers of story and fundamental periods of vibration were optimized subjected to a set of given earthquake ground motions using the concept of uniform damage distribution pattern along the height of the structures. Then, the mean ductility reduction factors subjected to 20 strong ground motions were computed and compared with those obtained from ASCE-7-16 [20] code-specified pattern assumption. Finalally, basedon regression analysis, a simplified expression which is a function of fundamental period and ductility demand to estimate ductility-dependent strength reduction factors of designed SMRFs according to code-based lateral load pattern is proposed.

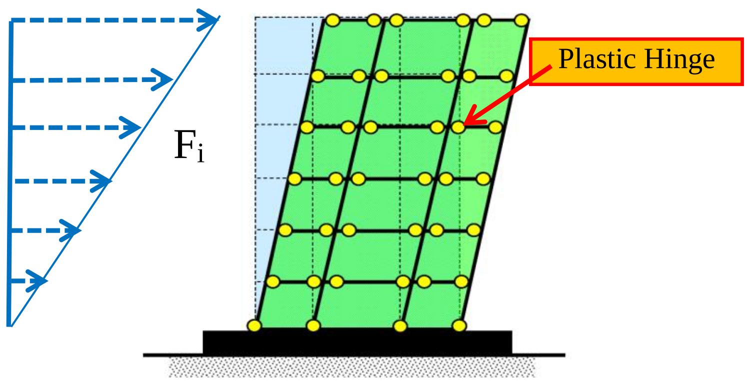


Figure 1. Schematic generic steel moment-resisting frame used in this study

2. GENERIC STEEL MOMENT-RESISTING FRAME USED IN THIS STUDY

For simplification and parametric analysis, during the past 20 years, different types of generic frames have been introduced and developed by many researchers for evaluating seismic response and behavior of steel and concrete moment-resisting frames. In geometry viewpoint, generic frames used in the past can be divided into two main categories: (1) fishbone-shape generic frames (2) single-bay generic frames, and (2). “Fishbone” shape generic frames are a type of generic frame utilized by Ogawa et al. [21], Luco et al. [22], Nakashima et al. [23], and Kahloo and Khosravi, [24]. In this simple model, a multi-bay frame can be modeled as a cantilever beam with two rotational springs at each floor level connected to roller supports on each side of the cantilever. One of the main assumptions in the development of this type of generic frames is existing the identical rotations of joints at the same floor. As with the second type of generic frame, many researches such as those conducted by Medina and Krawinkler [25], Esteva and Ruiz [26] and Park and Medina [11] showed that the response of a multi-bay building can be simulated adequately by a single-bay frame. 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]. In this study the single-bay steel frame reprehensive of steel moment resisting frame (SMRF) structures are utilized for parametric study. The capability of using this type of the frame will be evaluated and validated in the next section of this paper. A schematic shape of the generic frame is shown in Fig. 2.

The SMRF models used in this study regard to 10 single-bay, moment-resisting frames with the number of stories ranging from 3 to 20. The fundamental periods are 0.3, 0.6, 0.9, 1.2, 1.5, 1.8, 2.1, 2.4, 2.7 and 3.0 s. The main properties of the generic frames used in this paper are: Models are one-bay two-dimensional steel moment-resisting frames. The distribution of story mass is uniform over the floor levels. For all SMRF structures, story height is constant and equal to 3.6 m. Moreover, the beam span is equal to 7 m. The effect of finite joint regions is not taken into account, meaning the dimensions of centerline are considered for column and beam members. The generic frames are designed based on the strong column-weak-beam (SCWB) concept. 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. When the frame is undergone to a given lateral load pattern, the same value of over-strength 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. The first mode shape for all the models is a straight-line, which regards to the fact that each story stiffness is adjusted so 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 along the height is also achieved. 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 [25]. 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 bilinear elasto-plastic model with 3% strain hardening. It should be noted that a modified Rayleigh-type damping model for proper modeling of structural damping in inelastic plane structural systems proposed by Zareian and Medina [27] was utilized to have more reliable results in time history analysis.

3. SELECTING AND SCALING THE GROUND MOTIONS USED IN THIS STUDY

In this investigation, for nonlinear dynamic analyses an ensemble of 20 earthquakes ground motions was compiled from five strong earthquakes recorded on soil type D based on IBC-2015 [28]. They include 6 records of Loma Prieta earthquake with moment magnitude of 6.9, 4 records of Superstition Hills earthquake with moment magnitude of 6.7, 5 records of Northridge and 5 records of Superstition Hills earthquakes with moment magnitude of 6.7, and one record of San Fernando earthquake with moment magnitude of 6.6. They were selected from strong ground motion database of the Pacific Earthquake Engineering Center (PEER). These earthquake ground motions have been selected based on the following assumptions:

1. They exclude the near-fault ground motion characteristic such as pulse type and forward directivity effects.
2. They are not located on soft soil profiles; hence the effect soil-structure interaction has not been considered in this study.
3. They have no long duration characteristics.
4. The selected earthquake ground motions have moment magnitude equal or larger than 6.6 and closest distance to the fault rupture between less than 40 km.
5. These ground motions are recorded on soils that correspond to IBC-2015 [28] site class D, which is approximately similar to the soil type III of the Iranian seismic code of practice, Standard No. 2800 [29].
6. These ground motions have been scaled based on ASCE-7-16 [20] provision to be consistent with those that dominate 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. A sample of scaled ground motions for 15-story frame with fundamental period of vibration of 1.5 s is shown in Fig. 2.

4. VALIDATION OF SINGLE-BAY GENERIC FRAMES FOR COMPUTING THE STRENGTH AND DRIFT DEMANDS OF SMRFS

As mentioned in the literatures, previous studies showed that the introduced generic frames can have the capability of adequately accurate prediction of seismic demand parameters of interest. The generic frames used in this study are based on the assumption that they are capable to represent the behavior of more complex regular multi-bay frames. In this section it aimed at validating the seismic responses of a 9 story multi-bay steel frame of SAC building with those of its equivalent one-bay generic frame. To this end, the SAC LA9-M1 frame model [30] is considered to validate the utilizing the one-bay frames to adequately predict the response of regular multi-bay frames. The selected frame represents one of the steel perimeter moment resisting frames located in the north-south direction of a standard office building in the Los Angeles area, located on alluvium soil. Fig. 3 shows the elevation of SAC building frame used for validation. The selected frame is based on centerline dimensions with ignoring the geometry, strength, stiffness and deformations of panel zones effects. The force-deformation hysteretic behavior in plastic hinge locations is modeled by bilinear elasto-plastic behavior with 2% strain hardening. Global P-delta effect and the influence of the interaction between axial load and bending moment in columns was taken into account. 5% Rayleigh-type damping is assigned to the first mode and second mode. A one-bay frame model based on stiffness and strength properties of the SAC LA9-M1 model was developed according to the aforementioned simplified assumptions. More information can be found in Gupta and Krawinkler [30].

Modal and nonlinear static pushover analyses were performed to compare the responses of equivalent generic frame model with those of the multi-bay SAC frame. Based on modal analysis, the fundamental periods of one-bay generic frame and SAC frame are respectively 2.25 and 2.34 seconds, indicating that the basic modal properties are nearly similar. However, SAC model is slightly softer than the corresponding generic frame model. For the case of nonlinear behavior, a pushover analysis with a ASCE-7-16 load pattern [20] was performed. Fig. 3 is provided to compare the roof and first story pushover curves, respectively. As seen, although, the one-bay generic frame is approximately stiffer and stronger than the corresponding SAC frame, the general pattern and shape of the pushover curves is similar for the two considered models, indicating the yielding patterns and progressions are identical. The results of this study are consistent with those reported by Gupta and Krawinkler [30] and Medina [31].

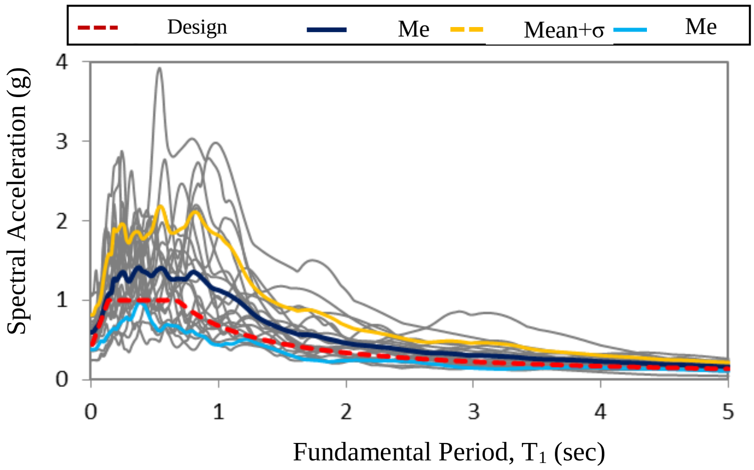


Figure 2. Ground motions scaled by ASCE-7 recommended approach [20] for a frame with T_1= 0.6 sec

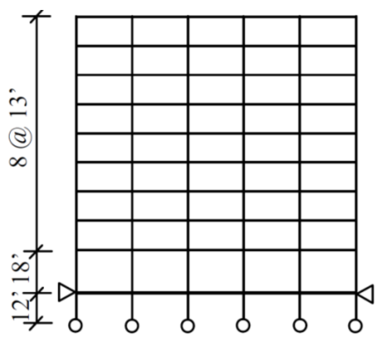


Figure 3. Perimeter Moment Resisting Frame of SAC building, LA9 -Ml model. [31]

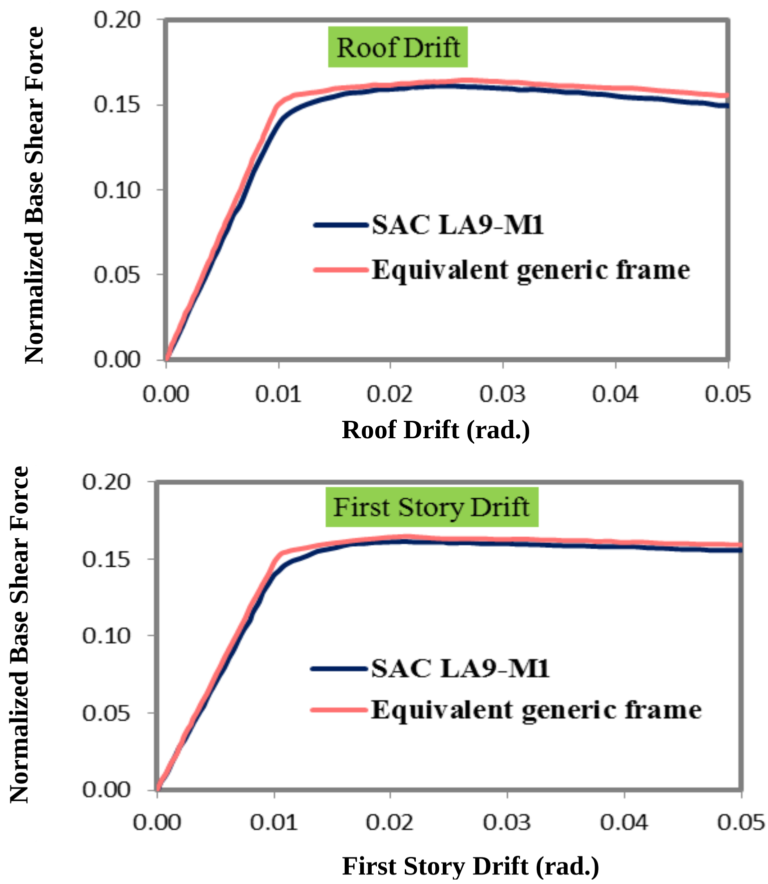


Figure 4. Comparison of roof (top) and first-story (bottom) pushover curves for SAC and generic-frame models

5. ESTIMATION OF DRF FOR OPTIMUM SEISMIC DESIGNED SMRFS

The main objective of this study is comparing the DRF values computed from code-based lateral load designed frames with those of optimum-designed counterparts. Many researchers made efforts to optimize various structural systems under static and ground motion excitations [15-17, 32-354]. In this study, Optimization algorithm utilized by Parka and Median [15] and then was modified by Ganjavi and Ghodarati Amiri [35] are utilized for estimating the DRF of optimum designed structures. Optimum-Designed SMRF models are regarded as the structure in which the story structural damage (i.e., ductility demand) are distributed uniformly along the height of a the frame under a given earthquake excitation. 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 in ASCE-7-16 [20] or Standard No. 2800 [29] seismic codes of practice. In such a case, one can easily compare the required elastic and inelastic strength demands of the code-based and optimum designed SMRF structures when subjected to a family of realistic earthquake ground motion excitations. In this regard, it is essential 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 [15, 16, 25, 31]. Two of the aforementioned parameters are widely used by many researchers to quantify the structural damage for non-deteriorating structural systems. The first parameter is the maximum inter-story drift ratio defined as the maximum relative displacement between two consecutive story levels normalized by the story height and the second one is inter-story ductility ratio defined as the maximum inter-story drift normalized by the inter-story yield drift.

$$\mu_i = \left(\frac{(\theta_{\max})_i - (\theta_y)_i}{(\theta_y)_i} \right) \tag{1}$$

where, μ_i is the inter-story ductility demand in *i*-th story. $(\theta_{\max})_i$ and $(\theta_y)_i$ are the maximum and yield drift angle of the *i*-th story, respectably. 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 two of the most appropriate parameters to determine the structural damage. It is believed that they have several advantages such as (i) they are very simple parameters to be computed by researchers; (ii) they are perceptible for all structural engineers; and (iii) 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.

In the present paper, the following step-by-step iteration process is proposed for the generic SMRF buildings under a given earthquake ground motion to achieve optimum-designed ductility reduction factor (DRF):

1. Define a generic frame prototype with specific number of stories, and select the target fundamental period (T_1).
2. 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. Consider the target inter-story ductility ratio, μ_t. In this study, the target values are 2, 4, 6, representing from low to high level of inelastic behavior.
4. Perform nonlinear pushover analysis and assign member strengths based on an arbitrary seismic design lateral force pattern such as code-based pattern. In this investigation, for nonlinear static pushover and non-linear time history dynamic analyses, the computer program DRAIN-2DX developed at the University of Berkeley [36] is utilized. The solutions are obtained using step-by-step integration of equations of motion using Newmark beta method.

5. Select and scale a given ground motion based on ASCE-7-16 [20] or 2800 seismic code [29] for the desired hazard level. Here, 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 is selected as hazard level.
6. Perform nonlinear dynamics time history analysis and calculate the maximum inter-story ductility ratio, $\mu_{max(i)}$. Control the ductility demand such that the following expression is achieved.

$$\beta_i = \left| \frac{\mu_{\max i -} \mu_t}{\mu_t} \right| \times 100 \le 0.5 \tag{2}$$

If the above condition is met, the structure will be regarded as optimum. Otherwise, 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 β_i is less than 0.5.

7. The designed frame is optimum and DRF can be computed from the following expression:

$$DRF = \frac{Ve\ (\mu_t = 1)}{Vy\ (\mu_t > 1)} \tag{3}$$

where V_e is elastic base-shear strength and V_y is inelastic base-shear strength corresponding to the target ductility demand (i.e., μ_t= 2,4,6).

8. The DRF values are computed for optimum and code-based designed patterns of other models having different number of stories, fundamental periods, ductility ratios and earthquake ground motions.

The step-by-step optimization algorithm proposed above is utilized to a 10-story building with T= 1.5 sec, and μ_t= 2 and 6 representing low and high levels of inelasticity subjected to 20 earthquake ground motions used in this study. Fig. 5 illustrates a comparison of the average results obtained from optimum designed structures and the corresponding code-compliant designed models. It can be seen that a significant difference is observed between the story damage distribution (ductility demand profiles) resulted from the two designed frames. In fact, the height-wise distributions of story ductility demands resulted from utilizing ASCE-7-16 [20] or Standard No. 2800 [29] design lateral load patterns [20] are very non-uniform with respect to the corresponding optimum cases.

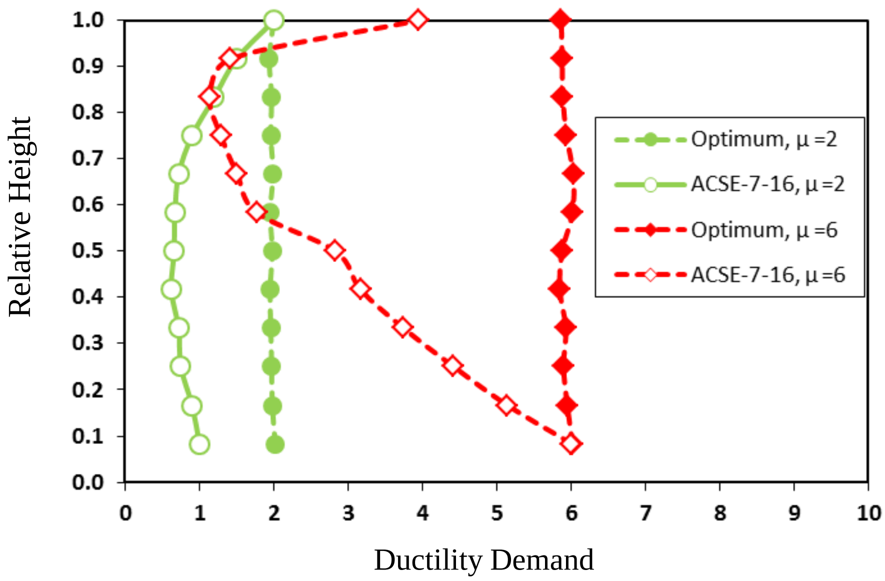


Figure 5. Story ductility profiles for optimum and code-based designed SMRF models, 10-story generic frame with T_1 = 1.5 s

By using the proposed optimization algorithm, the ductility-dependent strength reduction factors (DRF) were computed for the 10 generic-framed steel-moment resisting systems having the fundamental periods of vibration equal to 0.3, 0.6, 0.9, 1.2, 1.5, 1.8, 2.1, 2.4, 2.7 and 3.0 s, for three levels of inelastic behaviors μ= 2, 4, 6, representing the low, moderate and high inelastic states, respectively. The DRF values were computed for 20 earthquakes ground motions scaled by ASCE-7-16 provision to be consistent with those that dominate 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. Figs. 6 and 7 show the individual and mean spectra of ductility-dependent strength reduction factor (DRF) for optimum and code-based designed structures for different levels of inelastic behavior. As can be seen, the ductility-dependent strength reduction factors for both optimum and code-based designed structures are dependent on the fundamental period of vibration. However, the fundamental period affects the DRF value in a different manner. As shown in Fig. 6, for the case of short-period optimum designed frames, the DRF value increases with fundamental periods, whereas for longer-period frame models the mean value decreases inversely proportional to fundamental period of vibration. The results are different for code-based designed steel moment frames such that as observed in Fig. 7, the ductility-dependent strength reduction factors always increase as the fundamental period of vibration increases, implying that the ductility-dependent strength reduction can be considerably influenced by the height-wise distribution of structural characteristics such as story shear strength and stiffness.

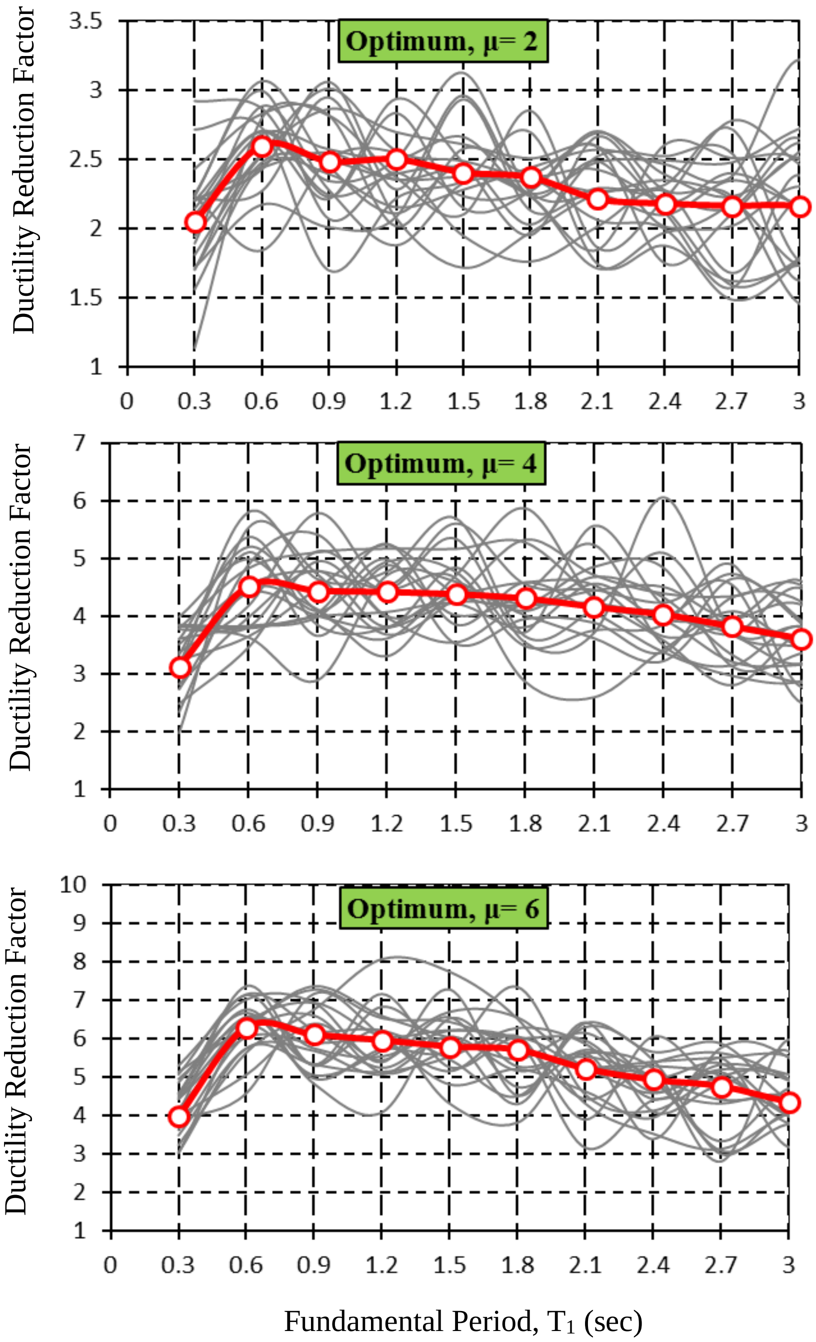


Figure 6. Individual (grey line) and mean (solid line) spectra of DRF for optimum designed structures

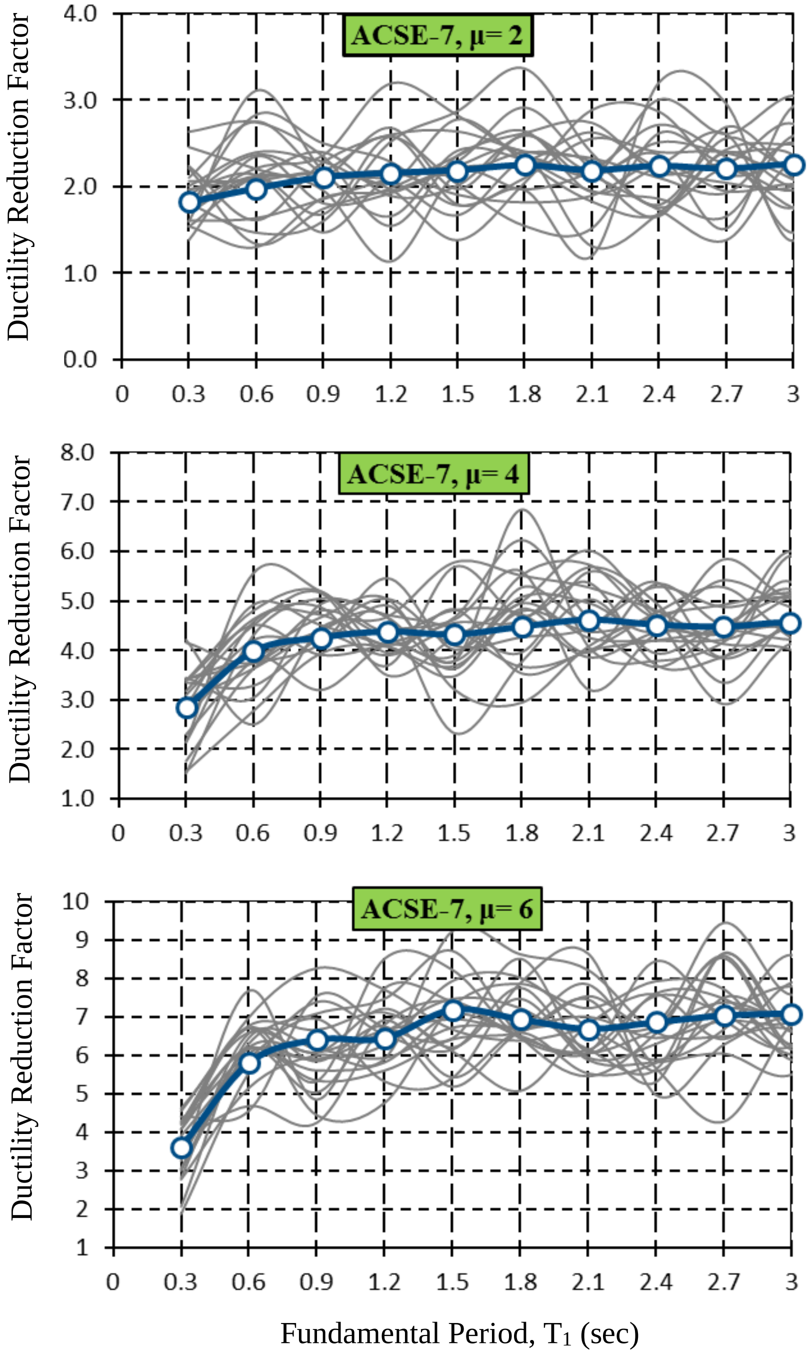
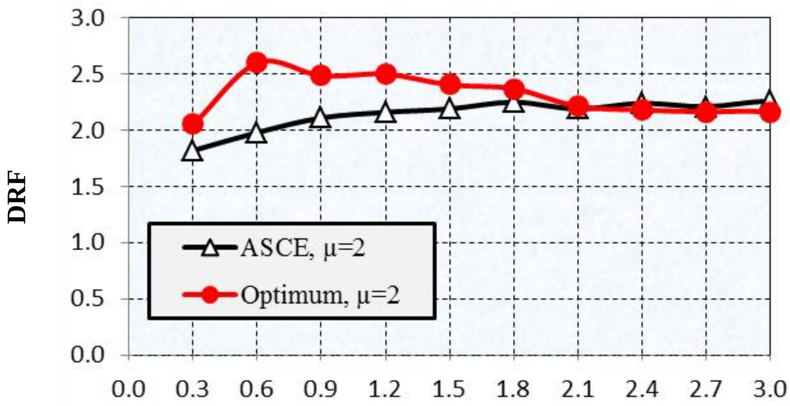


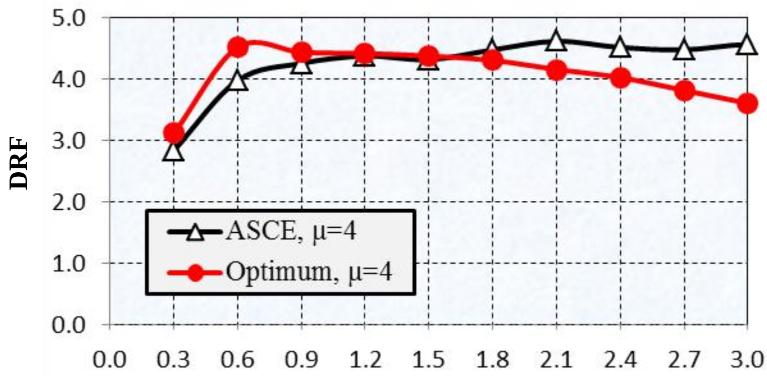
Figure 7. Individual (grey line) and mean (solid line) spectra of DRF for code-based designed structures

6. INFLUENCE OF OPTIMUM SEISMIC DESIGN ON DUCTILITY-DEPENDENT STRENGTH REDUCTION FACTOR

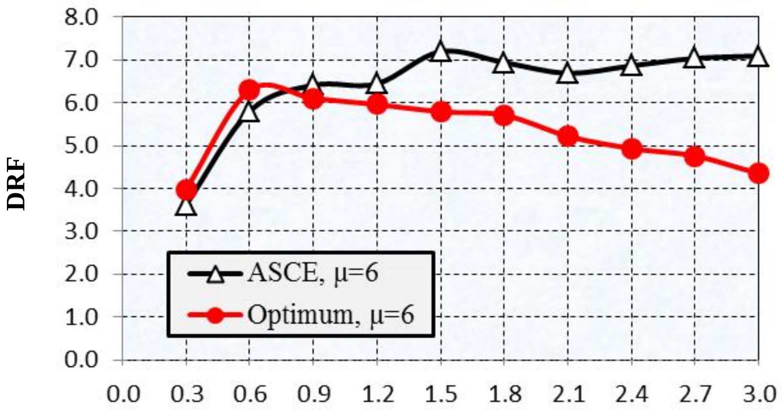
To more examine the effect of optimum damage distribution on ductility-dependent strength reduction factor spectrum, the mean values of DRF are separately depicted in Fig. 8 for different levels of ductility demand. Fig. 8a shows that for short- and moderate-period structures (T_1 equal or less than1.8 s) with low level of ductility demand (μ= 2), the DRF values of optimum structures are larger than those of the code-based counterparts. However, they are nearly converged to the same values for longer-period frames. This implies that the low- and mid-rise structures designed based on optimum load pattern with uniform damage distribution along the height require less seismic design base shear strength when compared to the frames designed based on the code provisions. As shown in Figs. 8b and 8c, by increasing the ductility demand, the DRF values become more sensitive to the variation of fundamental period. In fact, there is a threshold period before which the DRF values of the optimum designed structures are always larger than those of the corresponding code-based designed structures. Conversely, after that period, the DRF values of optimum designed structures are always lower than those of the corresponding code-based designed structures. The phenomenon is more intensified as ductility demand increases (see Fig. 8c for μ= 6). In addition, the threshold period is sensitive to the level of inelastic behavior such that it decreases as ductility demand increases. It can be concluded that, except for short-period structures, for moderate and high levels of inelastic demand the structures designed based on optimum load pattern with uniform damage distribution along the height require considerably larger seismic design base shear strength when compared to the frames designed based on the code provisions, which is more pronounced for long-period structures (i.e., the structural system becomes more flexible). For example, for a given target story ductility ratio of 6.0, the DRF for a frame with T_1 = 2.7 s. designed based on the ASCE-7-16 code provision is equal to 7.05 when compared to 4.76 for the corresponding optimum structure. This behavior can be associated to the P-delta effect tending to increase the story drift ratios of flexible structures, especially at the bottom stories.



(a) Fundamental Period, T_1 (sec)



(b) Fundamental Period, T_1 (sec)



(c) Fundamental Period, T_1 (sec)

Figure 8. Comparison of mean DRF spectra for optimum and code-based designed structures (μ= 2, 4, 6)

7. PRACTICAL EXPRESSION TO ESTIMATE DRF FOR SMRFS DESIGNED BY IRANIAN SEISMIC CODE LATERAL LOAD PATTERN

As mentioned in the literature, the code-based values of strength reduction factors in different seismic provisions are usually based on judgments, experiences and observed behaviors of structures during past earthquake events rather than on analytical results. In earthquake-resistant design, for practical purpose it is desirable to have a simplified expression to estimate ductility reduction factors of SMRFs. Here, based on nonlinear dynamic analyses of different generic steel frames designed by ASCE-7-16 load pattern [20] which is similar to that in standard No. 2800 [29], the equation proposed by Nasar and Krawinkler [9] for SDOF systems is modified for practical estimation of DRF for SMRFs as follows:

$$DRF = \left[(c(\mu - 1) + 1) \right]^{\frac{1}{c}} \tag{4}$$

$$c = \left(\frac{T_1^a}{T_1^a + 1} + \frac{b}{T_1} \right) \tag{5}$$

where T_1 is the fundamental period of vibration; *a* and *b* are constants depending on the inter-story displacement ductility ratio that can be obtained from Table 1. To show the capability of the proposed equation in estimating the ductility reduction factors of SMRFs Fig. 9 is provided. This figure shows the comparison of the proposed equation in predicting the ductility reduction factors of the frames with different ranges of nonlinearity obtained from Eqs. (4) and (5) with the averaged numerical results. As seen, there is a good agreement between prposed Eq. (4) and the averaged numerical results such that the *R*-squared values for μ= 2, 4, 6 are 0.98, 0.94 and 0.9, respectively.

Table 1: Coefficients a and b for Eq. 5

μ	*a*	*b*
2	-0.5	0.3
3	0.5	0.3
4	0.7	0.4
5	0.8	0.4
6	0.8	0.4

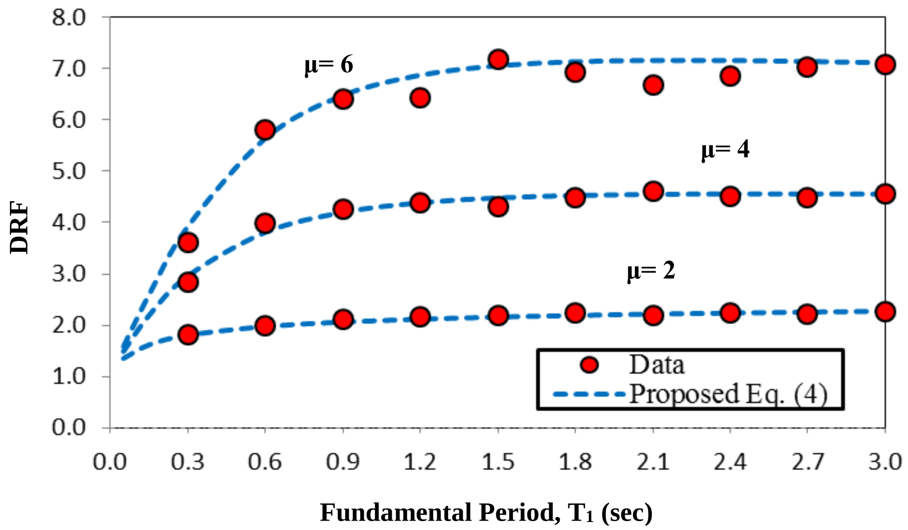


Figure 9. Correlation between equation (4) and averaged numerical results for ductility reduction factors of SMRFs designed by ASCE-7 or standard No. 2800.

8. CONCLUSION

In the present paper, 10 steel-moment resisting frames (SMRFs) having different numbers of story and fundamental periods of vibration were optimized subjected to a set of 20 earthquake ground motions using the concept of uniform damage distribution pattern along the height of the structures. Then, the mean ductility-dependent strength reduction factors (DRF) subjected to 20 strong ground motions were computed and compared with those obtained from code-compliant designed structures. Results of this study can be summarized as follows:

(1) The mean DRF values for both optimum and code-based designed structures are dependent on the fundamental period of vibration. However, the fundamental period affects the DRF value in a different manner. For the case of short-period optimum designed frames, the DRF value increases with fundamental periods, whereas for longer-period frame models the mean value decreases inversely proportional to fundamental period of vibration. For code-based designed steel moment frames, DRF always increases as the fundamental period of vibration increases, implying that it can be considerably influenced by the height-wise distribution of structural characteristics such as story shear strength and stiffness.

(2) There is a threshold period before which the DRF values of the optimum designed structures are always larger than those of the corresponding code-based designed structures. Conversely, after that period, the DRF values of optimum designed structures are always lower than those of the corresponding code-based designed structures, which is more intensified as ductility demand increases.

(3) It was found that, except for short-period structures, for moderate and high levels of inelastic demand the structures designed based on optimum load pattern with uniform damage distribution along the height require considerably larger seismic design base shear strength when compared to the frames designed based on the code provisions, which is more pronounced for long-period structures (i.e., the structural system becomes more flexible).

(4) Based on nonlinear dynamic analyses of different generic steel frames designed by ASCE-7-16 load pattern [20] which is similar to that in standard No. 2800 [29], an expression which is a function of fundamental period and inter-story ductility demand is proposed for practical estimation of DRF for steel moment-frame buildings. Results indicate that there is a good agreement between the proposed equation and the mean numerical data.

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