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Optimum Seismic Design of Short to Mid-Rise Steel Moment Resisting Frames Based on Uniform Deformation Theory

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ABSTRACT

In current work, an effective method is introduced for the optimal cross-section distribution in steel moment resisting frames under severe earthquakes by means of uniform deformation theory and adaptive method. The main goal is to distribute the construction material (weight) along the height of the structure in such a way that the lowest damage due to earthquakes is obtained. In adaptive method, materials gradually transfer from strong parts to weak parts by an iteration procedure during nonlinear time history analysis. In order to demonstrate the effectiveness of the proposed method, the optimal distribution of the cross-sections is obtained for 5 and 10 story steel moment resisting frames. In order to reduce the sensitivity of the optimal response to discrete cross-sections, continuous cross-sections fitted between DIN-Standard cross-sections have been used in order to achieve its optimal state. The steel moment resisting frames are optimized under five natural earthquakes. Results indicate that the optimal frames designed by this method show not only a more uniform deformation under earthquakes, but also less weight in comparison to the original structure designed according to the ASCE07-10 code. The reduction in structural weight reaches 40% in some cases leading to significant reduction in frame construction costs.

Keywords:
Seismic design; Optimization; Uniform deformation theory; Steel moment resisting frames; Nonlinear dynamic analysis

1. Introduction

In the conventional methods of seismic design, the distribution of lateral load along the structure height is usually determined using linear dynamic analysis. However, during severe earthquakes, the structure demonstrates nonlinear behavior and undergoes large displacements. Therefore, the linear response cannot represent the actual behavior of the structure during earthquakes, and consequently, the lateral load pattern proposed in the seismic design codes does not ensure optimal use of the materials in the structure. Previous studies by Moghaddam and Hajirasouliha [1] have shown that it is possible to distribute the structural material (weight) along the height of the structure such that the lowest damage due to earthquakes is obtained. In this regard, the theory of uniform deformation has been introduced to determine the way resistant factors are distributed within the structure. In this method, structural materials are transferred gradually from the parts with higher strengths to those with lower strengths through an optimization algorithm to achieve optimum distribution of the lateral load resisting elements within the structure. This continues until the distribution of deformation within the structure becomes uniform. It has been shown that seismic performance of a structure designed by this approach is optimized in the sense that the maximum capacity of the structure has been used [1]. Therefore, the above-mentioned optimization method has been utilized for the optimal seismic design of steel moment resisting frames and its effectiveness has been assessed and proven.

Researchers have proposed different optimization method for steel structures in recent decades. In these research, different optimization procedure such as dual method [2-3], evolutionary method [4], Lagrange method [5], and genetic algorithm [6-7] were used, and seismic excitation was modeled as a static force applied on the structure according to seismic codes. Then, cross sections of elements were designed optimally by means of optimization methods in linear systems. Thus, in these research, the dynamic effect of the seismic excitation and also nonlinear behavior of the frames was neglected, while in performance-based design for nonlinear structures, the use of seismic static force is an inefficient way and the structural performance parameter in earthquakes should be controlled directly.

On the other hand, researches represent that nonlinear static displacement control analyses (pushover) with a constant load pattern is not a reliable way for predicting damage in frame during earthquakes [8]. In this study, optimum seismic design of steel moment resisting frames by means of uniform deformation theory in time history analyses was investigated. The results show that with a proper distribution of material height-wise of the frame, damages in sever earthquakes could be in a restricted level and also the weight of the frames decreases by 40% weight of the customary designed according to static force of seismic codes.

2. Uniform Deformations Theory and the Adaptive Method

Moghadam and Hajirasouliha investigated the effect of distribution of resisting components within the structure on its seismic performance [1]. They modified the method presented by Karami Mohammadi et al [9] to resolve the fluctuations in the convergence procedure, and proposed a new method for the optimal seismic design of building structures. In this method, to achieve optimal distribution of resisting components within the structure, structural materials are transferred gradually from the parts with higher strengths to those with lower strengths through an optimization algorithm. This continues until the distribution of the deformation (or generally the damage) becomes totally uniform throughout the structure. It has been shown that the seismic performance of such structure is optimal, in which the maximum capacity of the structure is gained. Despite relative discrepancies between the optimization algorithms employed for different structural systems studied by Moghadam and Hajirasouliha [1], they all have the following fundamental steps:

1. First, the structure is designed according to an arbitrary distribution pattern for the behavioral parameters of the structure, based on which the preliminary design of the structure is done. These behavioral parameters can be the rigidity or strength of floors in shear structures, cross-section of truss members in truss structures, cross-section of braces in concentric braced frames, reinforcement percentage of the members in reinforced concrete moment resisting frames, cross-section of beams and columns in steel moment resisting frames, or any other factor that controls the behavior of the structure. Hajirasouliha and Moghadam [10] showed that the initial trial of the distribution pattern of the behavioral parameters does not have any effect on the final result.
2. The structure obtained in the previous step is subject to design loads. These loads can be either static or dynamic. Next, through performing proper iterative analyses, the behavioral parameters of the structure are scaled such that the structural design requirements are met, while maintaining their distribution pattern. The resulting structure at this stage is acceptable from a design perspective, yet might not be optimal. As this procedure continues, the damage demands of all structural components are obtained, and accordingly their coefficient of variations (Cov) are determined. If the calculated Cov is small

enough, the distribution of the (lateral) resisting components within the structure is considered to be optimized and the optimization procedure is terminated; otherwise, this procedure continues until an optimized distribution is obtained.

3. At this stage, the distribution of the lateral load resisting elements is modified according to the theory of uniform deformations. Simultaneously, the materials are transferred gradually to more critical regions from those that have not reached their full capacity. For this purpose, sections in which the damage demand parameter is lower than the limit (performance criteria) are specified, and their strength is reduced simultaneously. Investigations have shown that in order to achieve a proper convergence to the optimal response, these changes should be applied gradually [1]. To this end, to modify the behavioral parameters of the structure, the Eqs. (1) and (2) are used:

$$[(P_{SC})_i]_{n+1} = [(P_{SC})_i]_n (SC_i)^{\beta} \quad (1)$$

$$SC_i = \frac{dm_i}{dm_{ti}} \quad (2)$$

where, $(P_{SC})_i$ is the behavioral parameter of the ith member (the cross-section of members in truss structures, rigidity and strength of floors in shear structures, etc.), SCi is the convergence coefficient of the ith member, dm_i is the damage demand parameter required by the ith member (maximum displacement, maximum ductility factor, damage index, etc.), dm_{ti} is the objective damage demand parameter of the ith member, and n represents the iteration step number. β is also called convergence coefficient ranging from 0 to 1. Selection of the proper convergence coefficient highly influences the convergence of the problem and achieving optimal response. In this regard, by conducting numerous analyses on several structural systems and under different loading conditions, this coefficient has been determined for each case.

Using Eqs. (1) and (2), a new pattern is obtained for the distribution of lateral force-resisting elements within the structure. The optimization procedure is repeated from the second step so that another acceptable response is obtained. It is expected that Cov of the damage demand parameter of the elements at this stage is reduced significantly in comparison to the previous state. This trend continues until the Cov of the damage demand parameter becomes small enough so that a rather uniform distribution is achieved. The structure obtained at this stage is known to be practically optimal, in which the maximum capacity of materials has been used in it.

Although the preliminary method of Moghadam and Karami Mohammadi [11] uses a similar framework as the above-mentioned method, they are different in the sense that in the former, the gradual modification of the behavioral characteristics of members is not carried out systematically at stage 3, and the modifications are solely performed for the most critical member at every step. In other words, the level of changes in the behavioral parameter of the critical member across all steps is a percentage of its value in the previous stage. It has been shown that the preliminary method exhibits slow convergence and low accuracy that causes fluctuations in the convergence trend. In addition, the modified method presented by Hajirasouliha and Moghadam provides the possibility to choose different objective damage parameter for each structural element, allowing for seismic designs to achieve any desirable damage distribution pattern; while in the preliminary adaptive method by Moghadam and Karami Mohammadi, this was not possible [1, 11]. The advantages of the new adaptive method presented by Hajirasouliha and Moghadam include simplicity, the ability to automatize the method using rather simple programs, high convergence rate, suitable accuracy, lack of fluctuations in the convergence trend, allowing for achieving a unique response, allowing for seismic design to achieve any desire damage pattern, allowing for considering different load combinations in the structure design and simple development of the method for multi-criteria optimization [1].

3. The Optimization Algorithm of Steel Moment Resisting Frames

First, the preliminary structural model (that can be designed for gravity and seismic loads according to ASCE07-10 [12]) is subjected to seismic excitation. Accordingly, the plastic rotation of the end nodes of frame members is determined for the earthquake. The allowable rotation is calculated for those nodes according to ASCE-SEI41-06 [13]. Then, the beams and columns are altered in a way that the largest plastic rotation of every member approaches its allowable rotation. This means that if the maximum plastic rotation of a member is lower than the allowable value, the member becomes weak, whereas if this rotation is larger than the allowable value, the member is strengthened. Therefore, in order to design steel moment resisting frames optimally based on seismic loads, the following steps are considered:

1. A preliminary structure, previously designed using gravity and static seismic loads with a desired distribution pattern (here ASCE07-10 lateral load pattern) is regarded as the starting point. Within every iteration, the structure should be acceptable to the exerted gravity loads.
2. At this stage, the structure is subjected to seismic excitation and for the deformation controlled members including beams and some columns, maximum plastic rotation of each member (Θpi) and the allowable plastic rotation of that member (Θall) are determined based on ASCE-SEI41-06 regulations considering the life safety level (LS) as the objective structural performance. For this purpose, the allowable plastic rotation for the beams and columns is calculated using Eqs. (3) and (4) according to ASCE-SEI41-06, respectively.

$$\theta_{yb} = \frac{ZF_{ye} l_b}{6EI_b} \tag{3}$$

$$\theta_{yc} = \frac{ZF_{ye} l_c}{6EI_c}\left(1 - \frac{P}{P_{ye}}\right) \tag{4}$$

where, E, F_{ye} are the elastic modulus of the material and the expected yield stress; I_b, I_c, l_b, l_c and Z are the moment of inertia of the beam and column, the beam length, the column height and the plastic modulus of the cross-section, respectively. P and P_{ye} denote the axial force of the column, and the axial force of the yield limit expected in the column, respectively.

3. For brittle and force controlled members such as some columns, the force ratio that should be smaller than 1 is controlled by Eq. (5), according to ASCE-41-06. Furthermore, an error function is calculated based on the difference between the maximum and allowable plastic rotation, and the difference between the force ratio and 1 for deformation and force-controlled members, respectively. If the obtained error function is small enough, the distribution of resisting elements is assumed practically optimal and optimization procedure stops.

$$\frac{P_u}{P_{cl}} + \frac{M_u}{M_{cl}} \le 1 \quad , \quad \frac{P_u}{P_y} + 0.85\frac{M_u}{M_{cl}} \le 1 \tag{5}$$

where, P_u represents the axial force of the column, P_{cl} denotes the allowable axial force of the column, M_u is the applied moment of the column, and M_{cl} represents the allowable moment of the column.

4. At this stage, the cross-sections of members, representative of the stiffness and strength of frame elements, are modified. Using the theory of uniform deformations, the materials should be transferred from sections that have not been used in their full capacity to feeble parts of the structure. For this purpose, the cross-section of members whose maximum plastic rotations exceeds their allowable value should be increased, while the cross-section of members whose maximum plastic rotation is lower than their allowable value, should be reduced. Investigations have shown that in order to establish proper convergence, the variations within the structure should be gradual. Thus, cross-sections of members at every stage is modified according to Eq. (6):

$$[SN_i]_{k+1} = [SN_i]_k \left[\frac{\theta_{pi}}{\theta_{all}}\right]^{\alpha} \tag{6}$$

In this formula, SN_i represents cross-section of the i[th] member, k denotes the number of passed steps, and α is the convergence coefficient ranging from 0 to 1. Numerous analyses have indicated that this coefficient should be a small number so that the optimization trend continues slowly and uniformly. Larger α results in larger variations in the cross-sections of members in the next iteration and larger convergence rate, and vice versa. However, if this coefficient is chosen to be large, it is more likely for the optimization algorithm to show instability and divergence. This coefficient is chosen between 0.005 and 0.04 for members of steel moment frames. If the corresponding member approaches its allowable limit (for example the Θpi-to- Θall ratio is approximately 1), then the coefficient of α is chosen to be 0.005 in order to prevent the member to diverge from the allowable limit. However, if the demand of the corresponding member is significantly lower than its allowable limit, the power of α is chosen to be 0.04 to accelerate the convergence rate. Preliminary analyses show that the selection of a constant coefficient for the beams and columns in the 5-story frame can lead to convergence of the problem, while in the 10-story frame, divergence of the trend is observed and consequently the optimization algorithm is interrupted. This is because the resulting structure after the first iteration is too weak to be subjected to earthquake loads (for example, the cross-section of a member is too small). Furthermore, as the nonlinear analysis of this structure is not able to proceed, the optimization algorithm stops. Therefore, variable α coefficient has been used to prevent excessive variations in the members and in turn improper structures.

5. Next, in order to ensure that the frame can endure gravity loads, the frame is re-analyzed under gravity loads. If some members cannot withstand the gravity load, they will be strengthened gradually.
6. Using modified cross-sections, the optimization process is repeated again from the second step. It is expected that the error function in the new structure is lower than the corresponding value in the previous structure. The optimization operation is repeated until the error function becomes small enough and a relatively uniform distribution is obtained for the plastic rotations of the members.

4. Case Studies

To evaluate the optimization algorithm 5 and 10-story steel moment resisting frames were considered. Uniformly distributed dead load of 35.3 kN/m was assumed to be applied on all beams and uniform service live load was considered as 11.8 and 8.8 kN/m for floors and roof level, respectively. Figure (1) shows the 5 and 10 story frames [14].

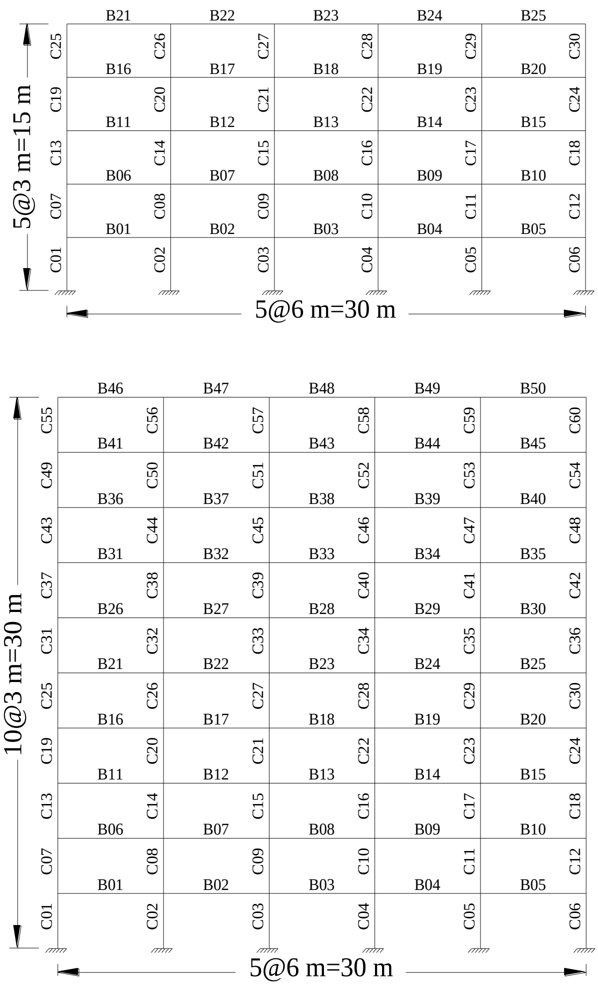

Figure 1. Finite element models for 5-bay moment-resisting steel frames with 5 and 10 floors.

To eliminate the over-strength effect in the design procedure, conceptual auxiliary sections were artificially developed by assuming a continuous variation of section properties. To achieve this goal, section dimensions (i.e. total height, flange width and web thickness) are approximated by exponential equations with respect to cross section, as the only effective parameter (as shown in Figure (2) for height of section). IPB and IPE sections, according to DIN-1025 standard, were chosen for columns and beams, respectively. All structural models should withstand the gravity loads. ASCE07-10 has been considered for gravity loads and ASCE07-10 lateral load pattern has been used to achieve a preliminary design of the frames. Note that it is an arbitrary and unnecessary assumption. The AISC360-2010 has been used for force controlled members, while the life safety (LS) performance level according to ASCE-SEI41-06 has been utilized for deformation controlled members. Since in the optimization algorithm the aim is to achieve the optimal structure, the inter-story drifts are neglected in the preliminary design [14].

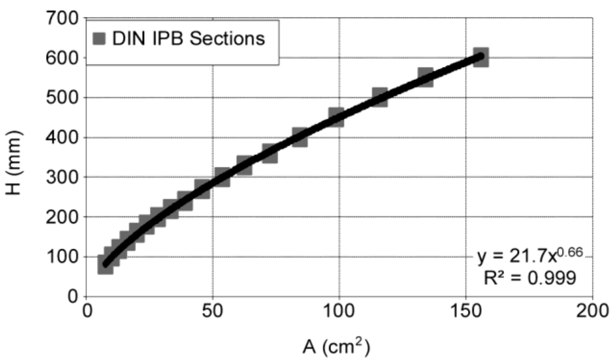

Figure 2. Exponential equation between H (height of the cross section) versus A (cross section).

5. Ground Motions

To investigate the efficiency of the proposed method, five medium-to-strong natural ground motion records were obtained from PEER ground motion database (Pacific Earthquake Engineering Research Center (PEER), 2000) as listed in Table (1). All of the selected records correspond to sites of soil profile C according to USGS, which is similar to soil type D of ASCE/SEI 7-10 (American Society of Civil Engineers (ASCE), 2010) and were recorded in a low-to-moderate distance from the fault rupture (between 5 and 15 km) with rather high magnitudes (i.e. Ms > 6.7). These records were used directly without being normalized [14].

6. Optimization Analyses

First, the preliminary design is obtained based on typical loadings of ASCE07-10 and according to the AISC360-10 specifications [15]. Then using a program previously written in MATLAB and by employing OpenSees to conduct the nonlinear analyses, the optimization algorithm starts. During the optimization procedure, the structure is initially analyzed under the earthquake loadings using OpenSees [16]. Afterwards, the analysis results are exported to MATLAB to provide OpenSees with a new structure (if any changes are required for the cross-sections) for analysis after assessing its structural performance. This procedure continues until achieving the optimal structure, where in every step the structure undergoes gravity load to ensure its adequacy. In order to investigate the efficiency of the presented optimization design method, 5 and 10-story steel moment resisting frames were optimally designed undergoing five natural earthquakes. The obtained results suggest that for all considered cases, the proposed algorithm results in reduction in structural weight as well as improved structural performance under seismic excitation. These results are shown in Figures (3) and (4) for the 5 and 10-story frame under Imperial Valley earthquake.

Table 1. Characteristics of ground motions [14].

EQ. #	Earthquake	Record/ Component	Station	Magnitude (Ms)	PGA (g)	PGV (cm/s)	PGD (cm)
16	Duzce, Turkey 1999	DUZCE/DZC270	Duzce	7.3	0.535	83.5	51.59
17	Imperial Valley 1979	IMPVALL/HE04140	955 El Centro Array #4	6.9	0.485	37.4	20.23
18	Loma Prieta 1989	LOMAP/G03000	47381 Gilroy Array #3	7.1	0.555	35.7	8.21
19	Cape Mendocino 1992	CAPEMEND/PET090	89156 Petrolia	7.1	0.662	89.7	29.55
20	Northridge 1994	NORTHR/NWH360	24279 Newhall - Fire Sta	6.7	0.59	97.2	38.05

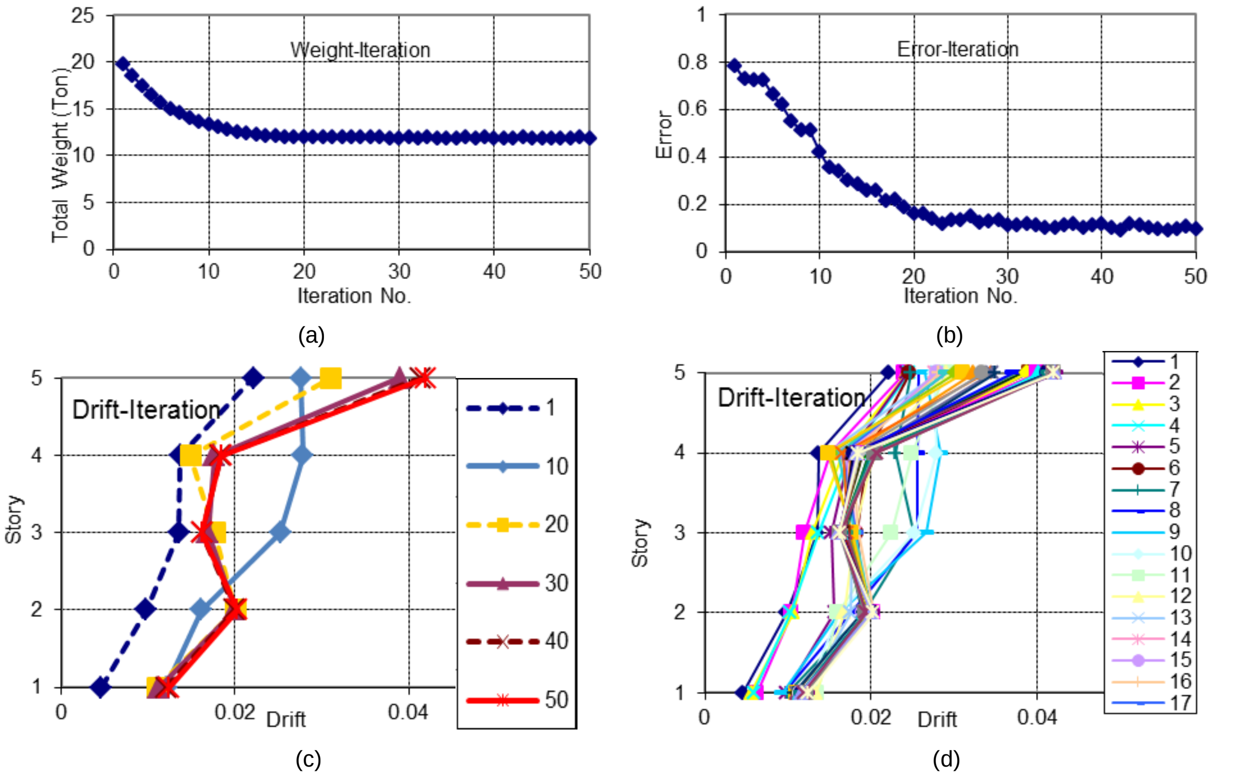

Figure 3. The results of the optimization of 5-stroy frame in Imperial Valley Earthquake, (a) changes of the structural weight across different steps, (b) variations of the error from the allowable value, (c) maximum inter-story drift of the frame every 10 steps, (d) the maximum inter-story drift of the frame.

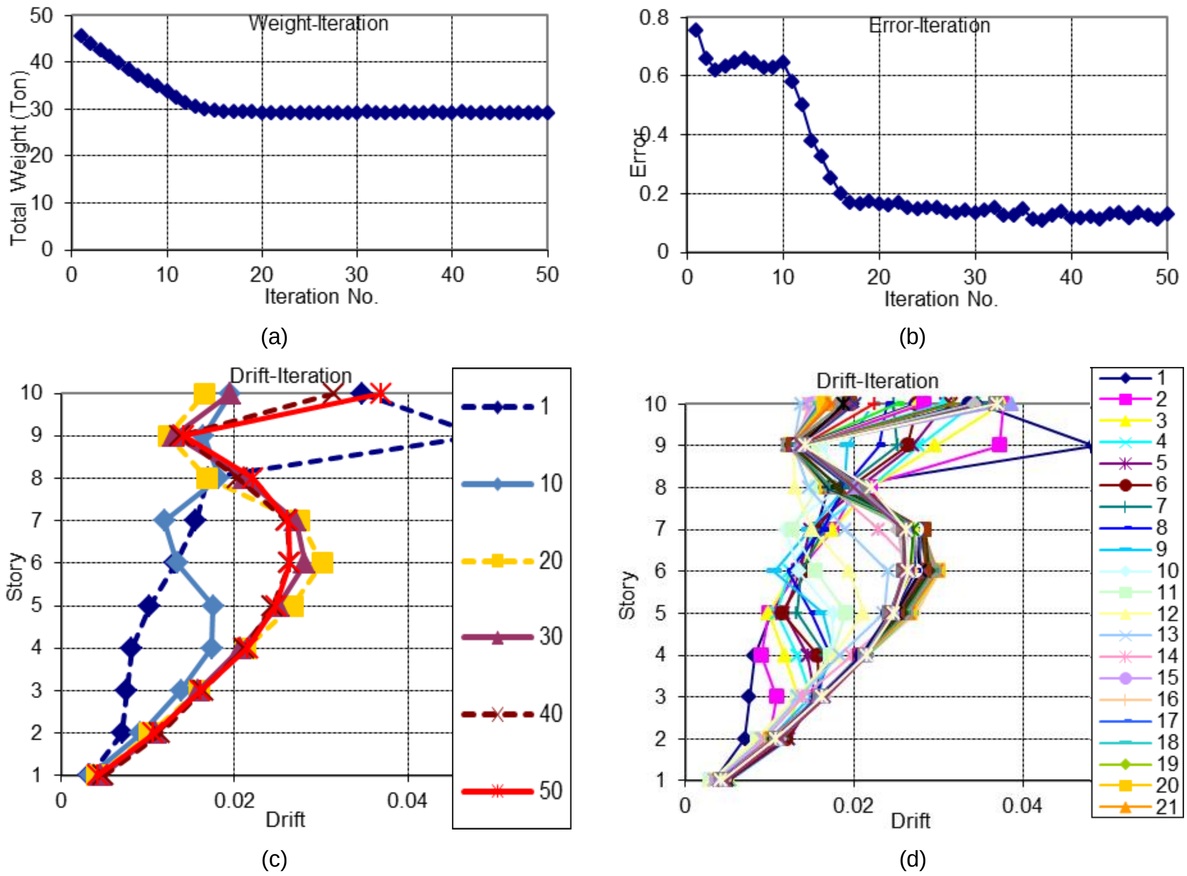

Figure 4. The results for optimization of 10-stroy frame in Imperial Valley Earthquake, (a) changes of the structural weight across different optimization steps, (b) variations of the error from the allowable value, (c) maximum inter-story drift of the frame every 10 steps, (d) maximum inter-story drift of the frame.

As shown in Figures (3a) and (4a) the variation in structural weight becomes negligible and tends towards a constant value. This is the optimum weight of the structure for that specific earthquake that depends on the earthquake intensity and characteristics. The negligible variation in the structural weight at the end of the diagrams shown in Figures (3a) and (4a) are due to the fact that the algorithm still tries to optimize the structure in the following iterations, but is only able to change the cross-sections to a little extent. Therefore, at the final iterations the structural weight fluctuates around the optimal cross- sections of the frame. These fluctuations can also be observed in terms of error variations (Figures (3b) and (4b)). The error function decreases from 0.75 to 0.1 at the end of the optimization procedure and it means that the performance criteria of the frame's elements approach LS limitation boundary of ASCE-SEI-4106.

Figure (3b) illustrates that the first frame's weight designed according to ASCE07-10 is 19.8 (ton), then at the end of optimization the weight decreases to 11.8 (ton). It means that the procedure reduces the weight by 40% and also the frame is in the Life Safety level according to ASCE-SEI-4106. Besides for 10-story frame, Figure (4b) shows that the frame's weight decreases from 45.7 (ton) to 29.2 (ton) (i.e. the frame's weight can be reduced by 36% of the first weight).

Figures (3c) and (4c) show that at the final iterations, the inter-story drifts also exhibit small variations due to slight change in the size or layout of the cross-sections. Therefore, the inter-story drift at the final steps also fluctuate a bit around the optimal inter-story drift. Results also show that as the structural weight reduces, the inter-story drift becomes more uniform. Therefore, the final structure has less weight and shows a better structural performance in an earthquake event. However, no constraint has been set to the inter-story drift in the optimization algorithm. This, itself, can be considered as a validation for the optimization algorithm.

According to ASCE-41-06, inter-story drift at LS level is determined to be 2.5% and in both 5 and 10-story frames, this performance criteria is satisfied for the optimized structures. For example, in the 5-story frame, the drift of the preliminary structure designed according to ASCE07-10, have been below 2% at each floor and the frame does not have a uniform drift across the floors. However, after reaching its optimal state, the inter-story drifts approach 2.5% across all stories and the drift distribution becomes relatively uniform across the floors. Thus it could be concluded that as the structural weight reduces, the inter-story drift becomes more uniform; therefore, the final structure has less weight and shows a better structural performance in an earthquake event. However, no constraint has been set to the inter-story drift in the optimization algorithm. This, itself, can be considered as a validation for the optimization algorithm.

The fluctuations in the optimization trend are mainly caused by force controlled columns and controlling of the entire structure under gravity loads, since one member at one iteration might be a deformation controlled member; while in another one it might be considered as a force-controlled member. In this situation, the algorithm tries to equate its maximum plastic rotation to the value allowed by the code, where the optimization parameter (i.e. ratio of maximum plastic rotation to the value allowed by the code) is 0.8, for example. At the next step, in order to increase the maximum plastic rotation of that member, the member's cross-section becomes weaker. When this happens, the member might not withstand the forces, thus the member is regarded as force controlled member. Afterwards, the optimization parameter (i.e. ratio of maximum forces applied to the value allowed by the code in this step) changes abruptly to 1.3 for example. Now in order to decrease the ratio of maximum forces applied to the value allowed by the code, the member's cross-section becomes stronger. Figure (5) demonstrates a sample of this type of variations in the cross-section of the columns in the first floor of the 10-

story frame. As it can be seen, due to the symmetry in the structure, the trend of C02 and C05 as well as C03-C04 and C01-C02 columns is relatively similar. Due to the fact that when the cross-sections become weaker, the column might undergo force controlled member's criteria, thus strengthening of the cross-section is required and a sudden increase in the cross-section strength might be observed.

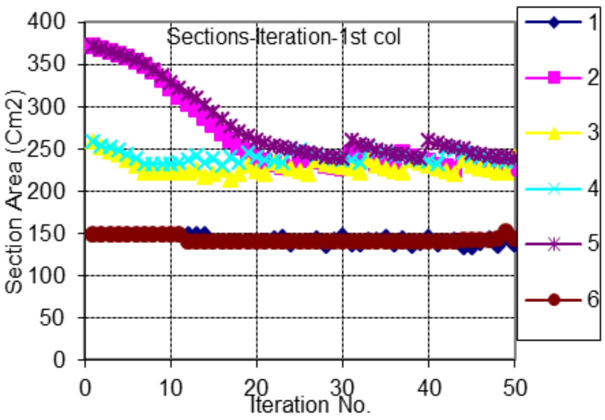

Figure 5. The trend of changes in the cross-section in the first-floor columns of the 5-story frame in Imperial Valley Earthquake in different steps.

Figure (6) also indicates that the cross-sections of the beams tend towards a constant value at the end iterations. Since the beams are deformation controlled members, the optimization algorithm tries to weaken them so that they exhibit more plastic rotation. However, the beams should be strong enough under gravity loads, so their cross-section cannot be reduced more than a certain level.

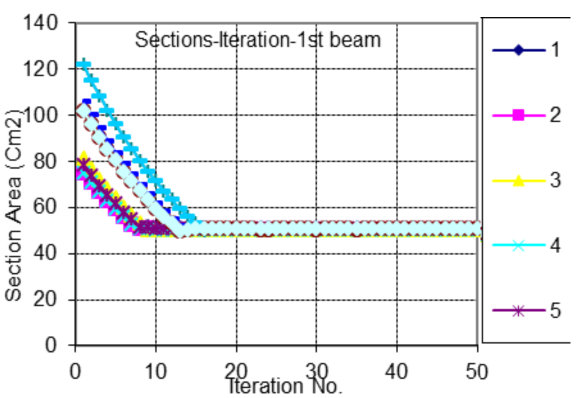

Figure 6. The trend of changes in the cross-section in the first-floor beams of the 10-story frame in Imperial Valley Earthquake in different steps.

Maximum drift ratio among stories in optimum frame during earthquake No. 5 is 6% occurs in 4th floor, during earthquake No. 4 is 5.2% occurs in 3th floor, during earthquake No. 2 is 4.2% occurs in 5th floor, during earthquake No. 1 is 4% occurs in 5th floor and during earthquake No. 3 is 3.3% occurs in 5th floor according to Figure (7). Therefore, the trend of maximum story drift ratios for 5 story frames is EQ.5> EQ.4> EQ.1> EQ.2> EQ.3 (although the maximum drift ratio of EQ.2 at 5th floor is larger than that for EQ.1 (i.e. 4.2 > 4) but average of the maximum drift ratios of EQ.1 is more than those for EQ.2). Thus it could be concluded that the optimum frame for EQ.5 has more weight than that for EQ.4, this is an obvious observation in Figure (8) in which the optimum frame weight in EQ.5 is 14.2 (ton), in EQ.4 is 14.05 (ton), in EQ.1 is 12.7 (ton), in EQ.2 is 11.8 (ton) and in EQ.3 is 11.2 (ton). Figure (9) also shows the trend of maximum drift ratio of 10-story frames as: EQ.1> EQ.5> EQ.4> EQ.2> EQ.3, this trend is also verifiable according to Figure (8) in which optimum frame weight in EQ.1 is 34.6 (ton), in EQ.5 is 33.6 (ton), in EQ.4 is 31.3 (ton), in EQ.2 is 29.1 (ton) and in EQ.3 is 28.4 (ton).

Since stronger earthquakes caused more damage and more drift ratios in stories during earthquakes, the optimization algorithm tend to increase the frames weigh (i.e. strengthen the frames) in order to withstand the severe earthquakes and maintain the performance level of the frames in LS level range.

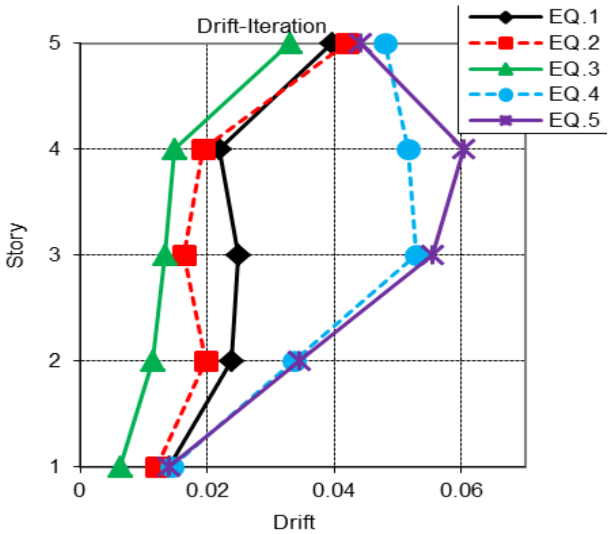

Figure 7. Maximum drift ratios of stories in optimum frame for different earthquakes.

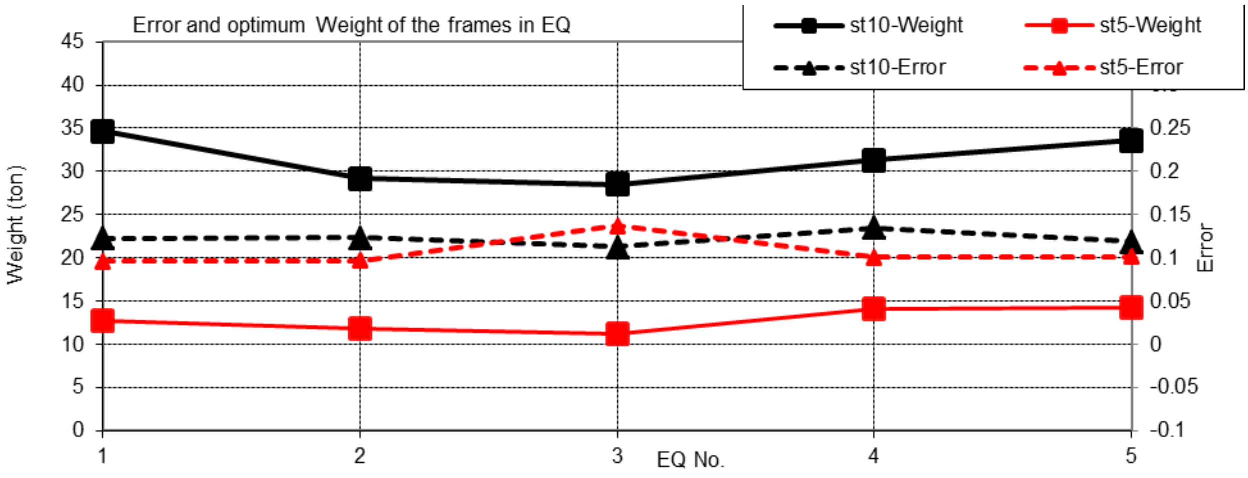

Figure 8. Final Error and optimum weight of frames in different earthquakes.

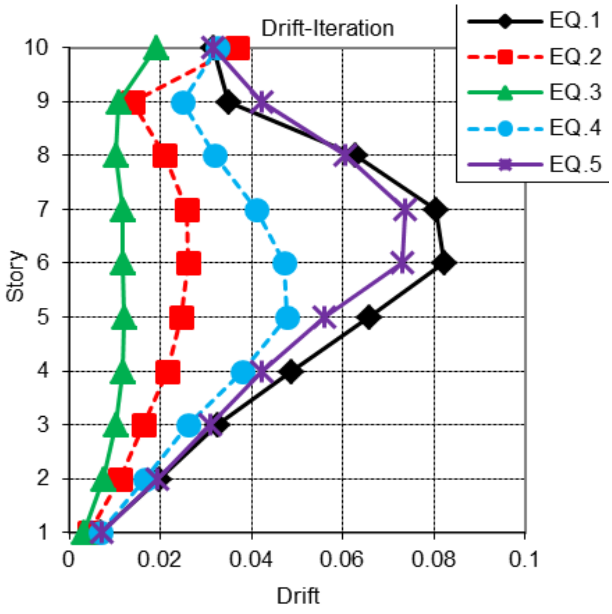

Figure 9. Maximum drift ratios of stories in optimum frame for different earthquakes.

Figure (8) shows the Error function value at the final iteration and the optimum weight of 5 and 10 story frames in different earthquakes. As it can be seen in all the cases for 5 and 10-story frames, the error function value reaches a small number (almost 0.1) and it means that the frame approaches to the LS performance level at the final iteration, and also the weight of the frame achieve its optimal value. The reason why the error function did not reach zero, is the fluctuation of the cross-sections around the final cross-section as well as variation of the category of columns from deformation-controlled to force-controlled members.

Average of maximum drift ratio of stories in optimum frame for five natural earthquakes versus maximum drift ratio of ASCE designed frame are shown in Figure (10). Although the drift ratios of the optimum frame are larger than drift ratio of ASCE designed frame but the structural performance level are still in LS performance level and the weight of the optimum frame is significantly less than those for ASCE designed frame.

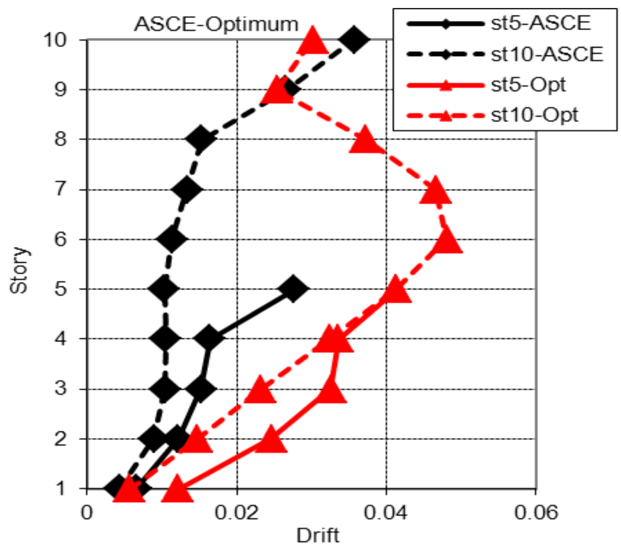

Figure 10. Average of maximum drift ratio of stories in optimum frame for 5 natural earthquakes versus maximum drift ratio of ASCE designed frame.

7. Conclusion

In current study, 5 and 10-storty steel moment resisting frames were designed according to the seismic codes and were afterwards optimized under five actual earthquakes. Results indicate that, the proposed method improves the behavior of the structure in terms of uniform inter-story drifts and reduction in structural weight for all the considered cases. It should be noted that in the optimized structures the plastic rotation performance criteria for the LS level of the ASCE 41-06 are satisfied.

It has been shown in this study that the method tend to increase the weight of the optimum frames in severe earthquakes, thus the optimization algorithm can easily take into the account the intensity of the imposed motion on the optimum frame. In this method for each earthquake the frames approach to an optimum weight in which the structural performance is also equal the one that designer was selected to be, so in this frame all of the materials are in optimum use in the frame. The optimization algorithm can also be used for static seismic loads and all other type of loads such as temperature, soil pressure, etc. either in static case or dynamic case with the corresponding constraints and desirable performance. In the proposed method, the structural performance was chosen to be LS level but one can easily change the desirable behavior of the frame to IO level or other performance criteria of stories such as drift, acceleration, etc. can be used.

The convergence coefficient, α, is an effective parameter in the optimization procedure. If a smaller number is chosen, the optimization trend moves towards the optimal structure with smaller fluctuations and slower rate. On the other hand, if a larger number is chosen, the optimization rate increases, but the fluctuations may increase to an extent that might disrupt the optimization procedure. Therefore, for the convergence coefficient of α for the steel moment resisting frame members, a range between 0.005 and 0.04 has been proposed depending on the difference between the optimization parameter and the allowable code based value, so that both the optimization algorithm rate is high and the divergence of the optimization trend is prevented. In this study, the frames approach their optimum state in only 20 iteration while other optimization algorithm such as genetic algorithm need a lot more iteration to approach the optimum frame, thus the proposed method is a strong and time-saving way to optimize the frame under seismic excitation.

One of the most important limitations of the proposed method is the sensitivity of the optimal response to the ground motion selection. In order to overcome this limitation, synthetic earthquakes according to the ASCE07-10 code spectrum can be used.

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