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OPTIMUM SEISMIC DESIGN OF STEEL MOMENT RESISITING FRAMES USING TIME HISTORY ANALYSIS

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Abstract

This paper presents the application of an innovative approach for performance-based design of structures. This approach is does not require a standard pattern for the distribution of seismic loads, and instead it is based on the principle of gradual adaptation of structural members to the seismic demand. The approach has already been implemented successfully for seismic design of various structural systems, and it has been shown that in most cases an optimum design can be reached accompanied by a uniform distribution of plastic deformations. The paper specifically discusses the application of the adaptation approach for the steel moment resisting frames (MRF). Five and ten storey MRF models are subjected to several seismic records. The reference model is designed in accordance with the seismic load pattern given in ASCE07-10. Other models are completely free to choose any distribution material to arrive at an optimal performance. The result is promising. It is demonstrated that the application of adaptation approach leads to a remarkable reduction in the weight of the structure at a given performance target. Alternatively, it can result in a considerable improvement of the performance, for a given structural weight.

Keywords: seismic design, optimization, uniform deformation principle, adaptation approach, steel moment resisting frames

1. Introduction

Traditionally, the seismic design is based on the evaluation of the total seismic forces applied to the base of a structure, base shear, and the heightwise distribution of this force. These are the two fundamental elements of seismic loading used worldwide since the early decades of the 20th century. However, during severe earthquakes structures demonstrate nonlinear behavior and undergoes nonlinear deformation. Once the structures are permitted to exceed the elastic range and undergo yielding, the seismic loading loses its purpose, and the conventional framework in the codes faces the-lack-of-rationale crises. Therefore, it is suggested that code requirements the base shear force and the heightwise pattern for the distribution of seismic loads could perhaps be regarded as imaginary and presumptive (1). It was shown that using patterns different from the standard one could sometimes lead to a better seismic performance (1). As discussed in the following section, in 1996 the first author developed an alternative approach for aseimic design. This approach employs the principle of uniform deformation, where the strength is distributed within the structure in a manner to arrive at a status of uniform deformation. This ensures that the design is optimal. The problem with the optimum design is that in a real structure there are a huge numbers of possible distribution patterns to be checked for arriving at an optimal design. This could be time consuming and impractical. The beauty of the proposed approach is that it narrows down the wide spectra of all potential candidates very quickly and efficiently similar to the way that the nature does in the process of natural selection and adaptation. The approach has been employed so far for optimum design of various types of structures (2-8).

2. Moghaddam's Approach

In both conventional and performance-based designs, it is required to use some specific patterns for distributing the seismic loads. In the conventional design, the pattern is usually a triangular. In the push over analysis, in addition to the triangular, it is required to use a second pattern which could be the first mode of vibration. Moghaddam [1] argued that these conventional patterns may not necessarily lead to the best performance. He showed that better results are obtained by using different patterns. He also developed an approach where there was no need to use seismic forces for aseismic design. Moghaddam's approach is inspired by the natural adaptation of the body where those parts that are under more pressure become stronger and vice versa. This approach can be outlined as follows [1-8]:

1. The structure is designed for the gravity loads regardless of seismic forces.
2. A nonlinear dynamic analysis is performed subjecting the structure to the seismic excitation.
3. The members are modified using the following equation.

$$(P_{SC})_1 = (P_{SC})_0 F(\frac{\delta}{\delta_t}) \tag{1}$$

Where P_{sc} is the parameter that best controls the seismic response of the member. Subscripts 1 and0 correspond respectively to the present and previous cycles of analysis. The function F serves as an adaptation factor. It depends on the ratio $\frac{\delta}{\delta_t}$, where δ and δ_t are the calculated and target deformations, respectively. The control parameter P_{sc} depends on the structural function of the member within the seismic resistant system. For example, for shear buildings, the story shear strength is usually considered as the control parameter [4], while for braced structure, it is the cross section of braces that predominantly controls the response, and therefore, has been used as the controlling parameter [4,5]. The bending reaction of beams and shear walls depends on parameters such as initial moment of inertia and bending strength which can be used as P_{sc} [3, 7]. The adaptation function can be theoretically any mathematical function that helps closing the gap between demand and capacity deformations, δ and δ_t by modifying the member properties. A number of such functions have been developed [3, 4]. A general feature of such functions is that they should close the foregoing gap very slowly to avoid instability in analysis. An example of such functions which has been used widely for many types of structural members is as follows [5]:

$$F = \left(\frac{\delta}{\delta_t}\right)^{\alpha} \tag{2}$$

The parameter α controls stability and convergence. While an appropriate value of α depends on the type of structure, considering a low value of 0.1 usually guarantees both stability and convergence in many cases.

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 [9]) 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 [10]. 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 (Θ_{pi}) and the allowable plastic rotation of that member (Θ_{all}) 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 equation 3 and equation 4 according to ASCE-SEI41-06, respectively.

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

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

   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 equation 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 principle 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 equation (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 iterations, 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 Θ_{pi}-to- Θ_{all} 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 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 isn't 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. Fig. 1 shows the 5 and 10 story frames [11].

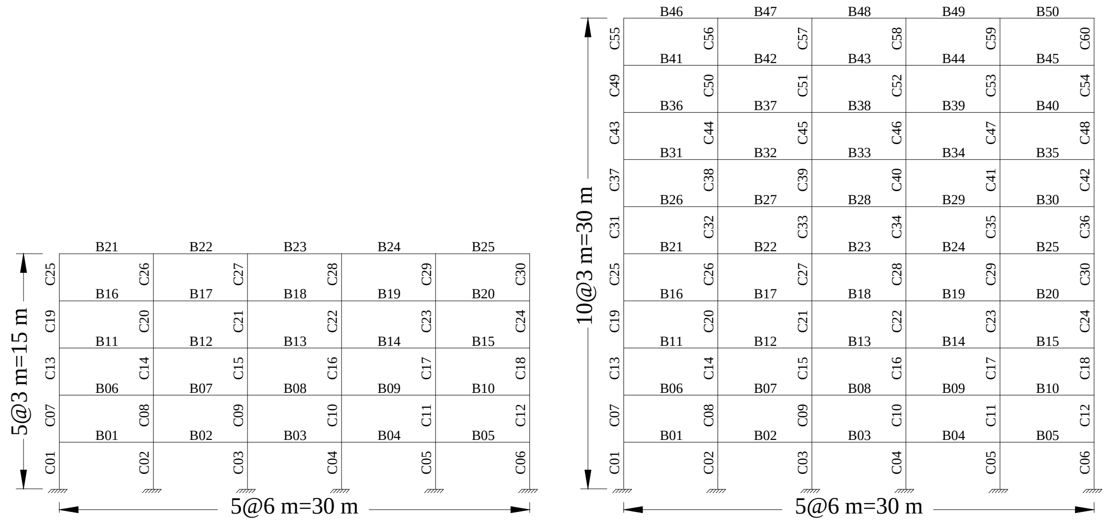


Fig. 1. Finite element models for 5-bay moment-resisting steel frames with 5 and 10 floors

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 Fig. 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 [12] 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 [11].

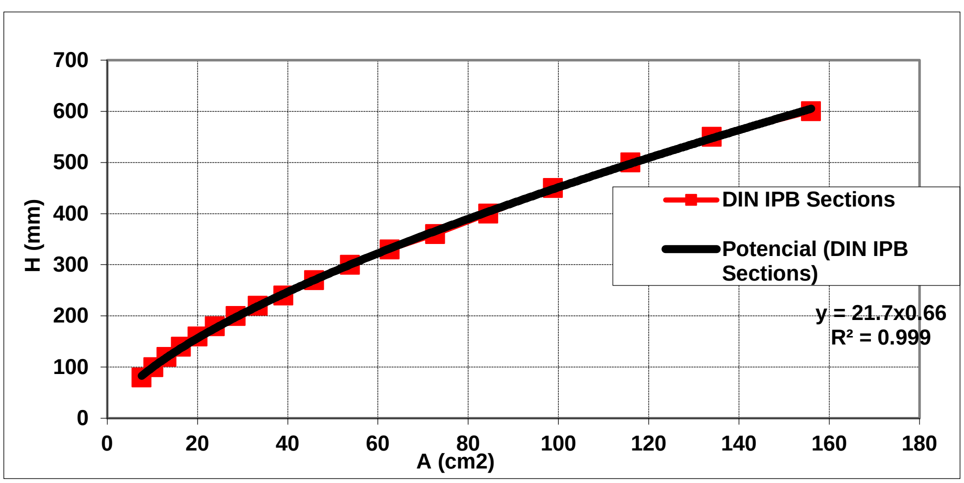


Fig. 2. Exponential equation between H (height of the cross section) versus A (cross section).

5. Ground Motions

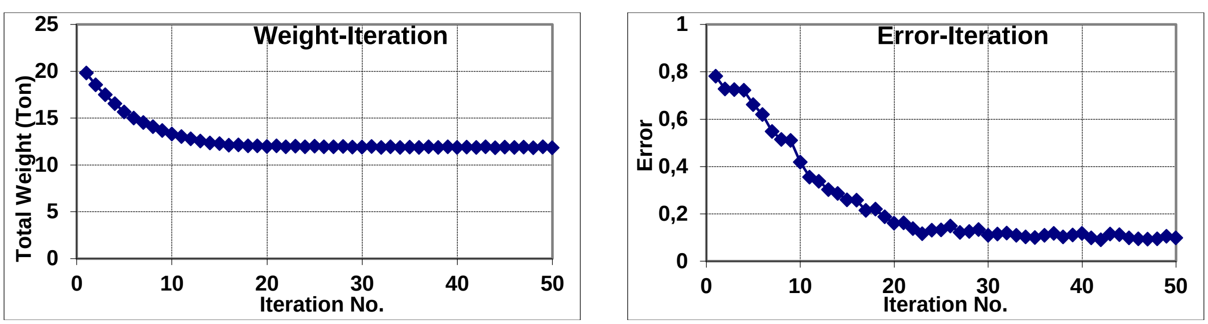
To investigate the efficiency of the proposed method, five medium-to-strong natural ground motion records were obtained from PEER ground motion database (Pacific Earthquake Engineerng Research Center) as listed in Table 1. All of the selected records correspond to sites of soil profile C according to USGS, which is similar to soil type D of ASCE/SEI 7-10 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 [11].

Table 1. Characteristics of ground motions [11]

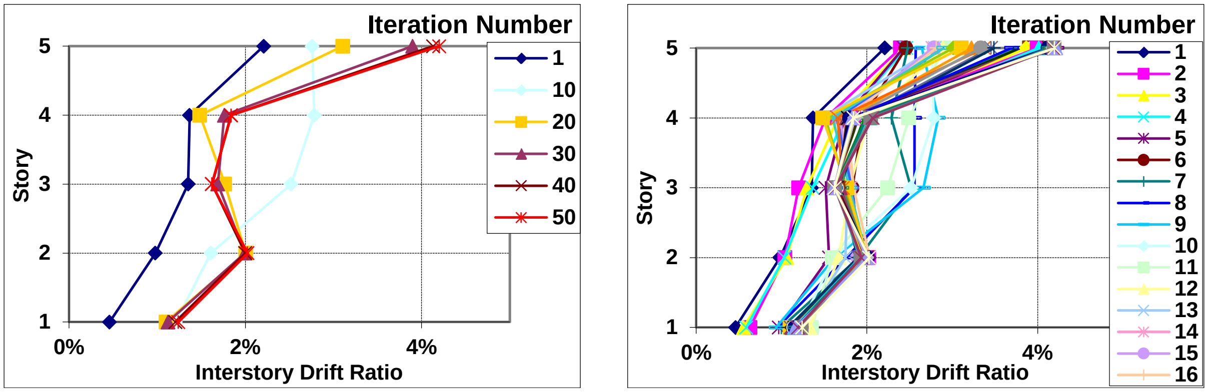
EQ . #	Earthquake	Record/ Component	Station	Magnitude (Ms)	PGA (g)	PGV (cm/s)	PGD (cm)
16	Duzce, Turkey 1999	DUZCE/ DZC270	Duzce	7.3	0.535	83.5	51.59
17	Imperial Valley 1979	IMPVALL/ HE04140	955 El Centro Array #4	6.9	0.485	37.4	20.23
18	Loma Prieta 1989	LOMAP/ G03000	47381 Gilroy Array #3	7.1	0.555	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

6. Optimization analyses

First, the preliminary design is obtained based on typical loadings of ASCE07-10 and according to the AISC360-10 specifications. Then using a program previously written in MATLAB and by employing OpenSees to conduct the nonlinear analyses, the optimization algorithm starts. NonlinearBeamColumn element with distributed plasticity was used for modeling beams and columns and nonlinear geometric considered by using corotational geometric transformation in opensees. During the optimization procedure, the structure is initially analyzed under the earthquake loadings using OpenSees [13]. 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 iteration 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 5 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 Figs. 3 and 4 for the 5 and 10 story frame under Imperial Valley earthquake.

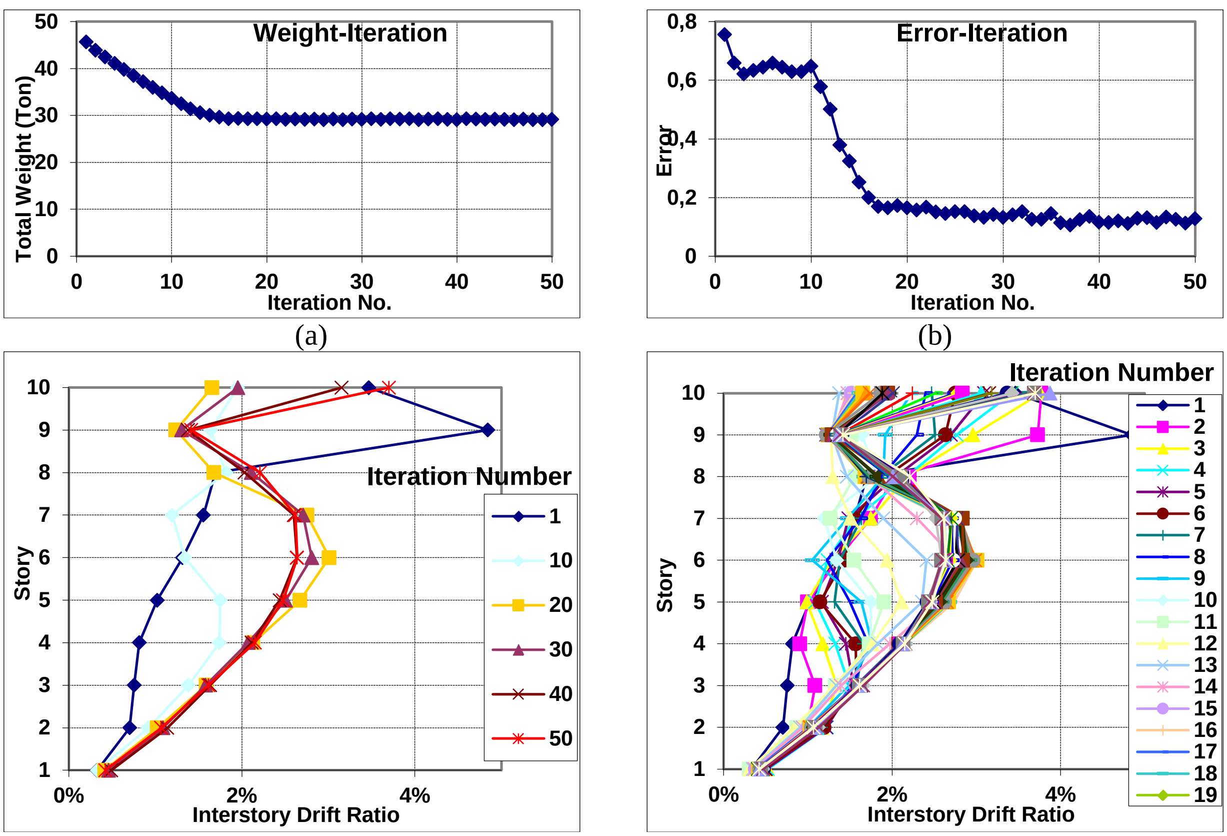


(a) (b)



(c) (d)

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



(a) (b)

(c) (d)

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

As shown in Figs. 3(a) and 4(a) 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 Figs. 3(a) and 4(a) 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 (Fig. 3(b) and 4(b)). The error function decreases from 0.75 to 0.1 both for 5 story and 10 story frames at the end of the optimization procedure and it means that the performance criteria of the frames' elements approach Life Safety limitation boundary of ASCE-SEI-4106 structural performance.

Fig. 3(b) illustrates that the first 5 story frame's weight which is designed according ASCE07-10 is 19.8 (ton) then at the end of optimization the weight decreases to 11.8 (ton) it means the procedure reduces the weight by 40% and also the frame is in the LS level according to ASCE-SEI-4106. Also for 10 story frame Fig. 4(b) 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). Figs. 3(c) and 4(c) 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 iterations also fluctuate a bit around the optimal inter-story drift. 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 floors 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. So 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 iteration, 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 iteration) 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. Fig. 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.

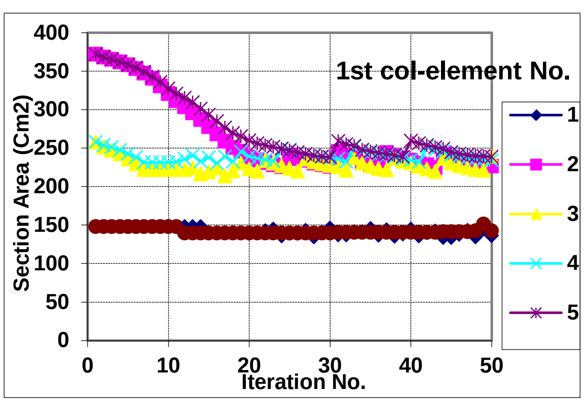


Fig. 5. The trend of changes in the cross-section

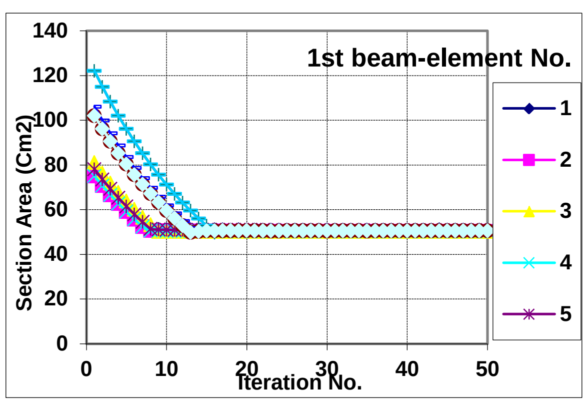


Fig. 6. The trend of changes in the cross-section

in the first-floor columns of the 5-story frame in Imperial Valley Earthquake in different iterations

in the first-floor beams of the 10-story frame in Imperial Valley Earthquake in different iterations

Fig. 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.

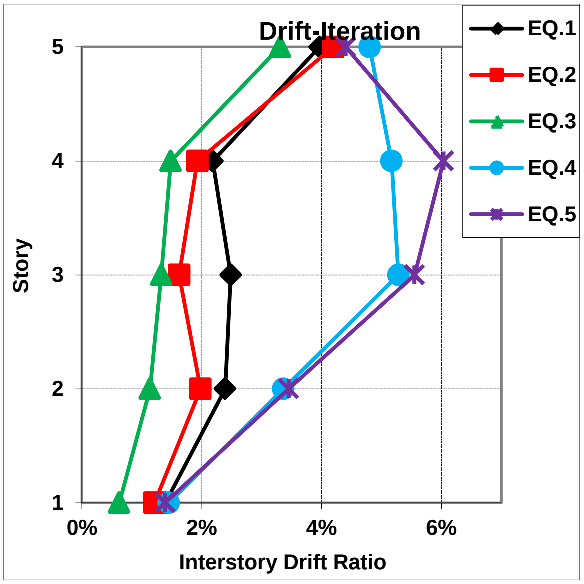


Fig. 7. Maximum drift ratios of 5 story frame in optimum state for different earthquakes

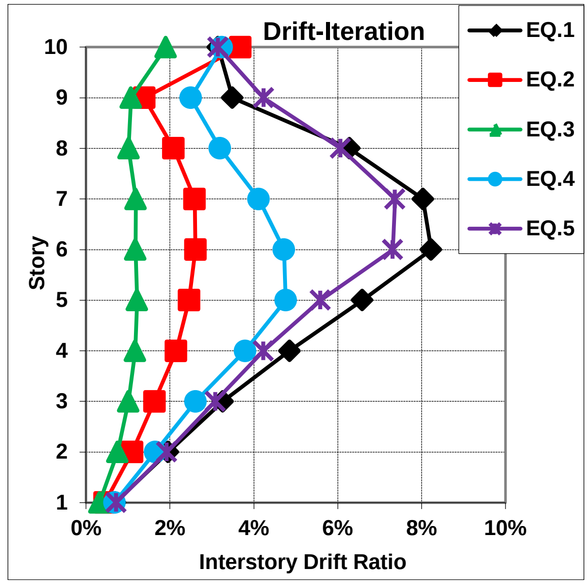


Fig. 8. Maximum drift ratios of 10 story frame in optimum state for different earthquakes

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 Fig 7. So 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 Fig 9 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). Fig 8 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 Fig 9 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.

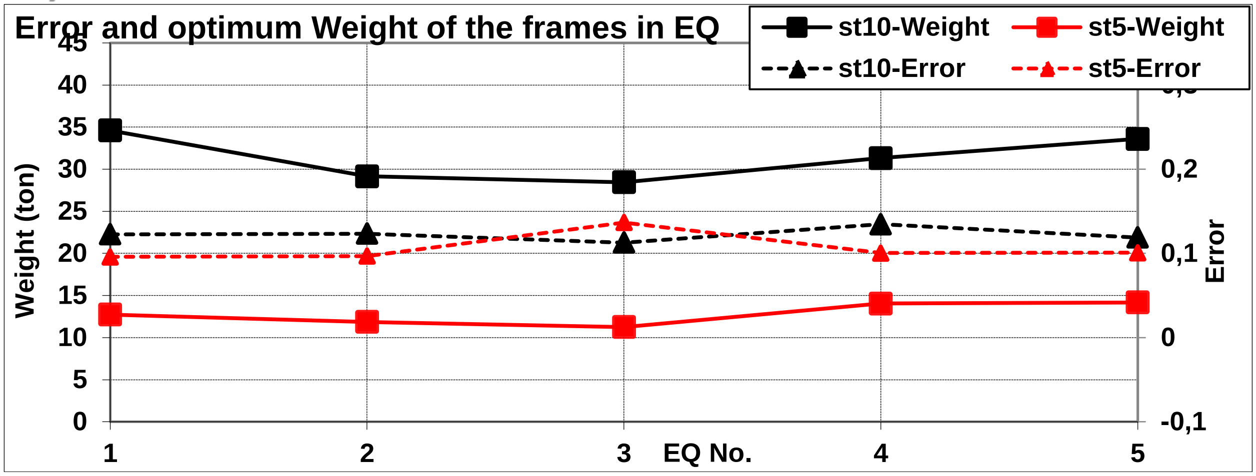


Fig. 9. Final Error and optimum weight of frames in different earthquakes

Fig. 9 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.

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 5 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, so 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. 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 and diverge 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 to 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, So 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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