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OPTIMUM SEISMIC DESIGN OF STEEL MOMENT RESISTING FRAMES BY GENETIC ALGORITHMS

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Abstract

Applying elastic methods in the design of steel moment resisting frames (SMRF) and not recognizing the redistribution of moments in the inelastic range, do not guarantee a suitable seismic behavior in earthquakes. In order to be able to predict and control the inelastic behavior under seismic loading and to determine the corresponding load factor, the design of SMRF is studied in this paper. Classic concepts of plastic analysis and genetic algorithms are combined to arrive at an optimal proportioning of the frame members. Various examples, along with studies on the parameters of the employed genetic algorithm are also presented within this work.

Keywords: Steel moment resisting frames; seismic design; yield mechanism; plastic analysis and design; genetic algorithm

1. Introduction

Steel moment resisting frames are considered as one of the most suitable structural load bearing systems for carrying gravitational and seismic loads, by structural engineers. These structures have often been designed using elastic methods. However, earthquake engineers expect inelastic behaviour of such frames during severe earthquakes. In addition, after several severe earthquakes, which occurred in different parts of the world, the shortcomings of the prevalent design methods have become apparent to structural experts and have encouraged them to search for more efficient design approaches [1-3]. Many studies have been carried out on these structural systems and some efficient methods have been proposed [4-12]. These studies have shown that the best performance among moment frames belongs to the ones which were designed based on the strong column-weak beam philosophy [13-18]. Thus, design philosophy of building codes has gradually moved toward these concepts and some emerging design formulae appear to have built confidence in the proper behaviour of structures under seismic loading.

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In addition to the lessons learned from recent earthquakes, the results of various theoretical and experimental studies revealed that using these relationships is not sufficient, as these formulae do not consider the redistribution of moments in the post elastic range [15,18]. In this paper, an efficient approach is proposed, which is based on the method of combined plastic mechanisms and a real genetic algorithm. In the present method, originally a performance-based design approach, the objective is to arrive at a suitable collapse mechanism under seismic loading. Another factor taken into account is requirement to make use of a minimum amount of material, which can be achieved by considering a unit collapse load factor for the desirable mechanism. A genetic algorithm with real design variables is used as a means of design procedure.

2. Seismic Design Philosophy and Objectives

Nowadays, many different building Codes of Practice have similar goals for seismic design of structures and only differ in their details. In most cases building codes assume that the buildings will not experience structural- or non-structural damages during the weak earthquakes that may occur several times during their lifetime. The structures should also be able to remain stable in such a manner that the people's lives can be saved during a single severe earthquake, which is considered a rare occurrence. In the severe earthquakes that happened in the recent decade, like the Northridge earthquake in California (1994), the Cobe earthquake in Japan (1995), the Taiwan earthquake (1999) and the Central Western earthquake in India (2001), although engineered structures have been able to save lives, the extensive financial damages and the extent of the physical destruction revealed the weaknesses of current design methods [1-3]. This has inspired extensive efforts by researchers to find more rational criteria for the seismic resistant design of structures [4-7]. In most of these methods the strengths and deformations in the design approach are considered simultaneously [8-12]. Currently, many seismic building codes are moving toward performance-based design methods. In comparison to previous methods this approach not only has a rational foundation, but can also control the structural behaviour and damage during earthquakes in a desirable manner. Many behavioural parameters, such as ductility and storey drifts, can be measured to evaluate the performance of the structure. The method presented in this paper can be considered a performance-based approach, since, in addition to controlling lateral deformations, it allows the designer to determine the collapse mode and the corresponding load factor under seismic loads, recommended by codes of practice. The general seismic design philosophy allows the structure to enter into its nonlinear region. Consequently, plastic hinges can be formed in some locations without violating global structural stability and safety. Moreover, design forces are determined by considering the effect of hinge formation on the seismic input energy dissipation.

Even though a structural designer can choose whether the columns or the beams should yield first, it is generally desirable to provide strong columns and to allow prior yielding of the beams in flexure. The reasons for this choice are as follows:

1. Column failure means the collapse of the entire building.

2. In a weak column structure, plastic deformation is concentrated in a certain storey, as shown in Figure. 1. Consequently, a relatively large ductility factor is required.
3. In both shear- and flexural failures of columns, degradations are greater than when beams yield. This is due to the presence of axial forces in the columns, Wakabayashi [19].

In Figure 1, a desirable and an undesirable collapse mechanism are shown. In the structure designed with strong columns and weak beams, plastic hinges will ultimately be formed at the base of the first storey columns. Hence, sufficient ductility should be provided for columns. This will be discussed further in the subsequent section.

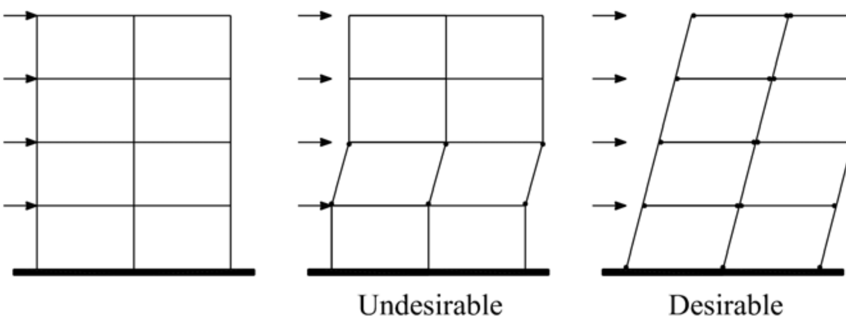

Figure 1. A frame and the corresponding undesirable and desirable mechanisms

3. A Review of the Previous Studies

After the Northridge (1994) and Cobe (1995) earthquakes, many damages in beam-to-column joints and numerous other damages in steel moment resisting frames have been reported. These damages show that there is not enough accurate information available on seismic behaviour of buildings and further in-depth studies are needed. As a result, no one can claim with certitude how safe current frames are, or how existing weak frames should be rehabilitated and whether new SMRFs should be built.

Various theoretical and experimental studies on the effect of using codes on the behaviour of SMRFs have been performed. Although most of the studies in the literature are based on the previous design codes, judgments based on the results of these studies can be made for the new SMRFs, since the main concepts related to this matter have not changed considerably.

Previous studies on the seismic behaviour of SMRFs can be divided into theoretical and experimental categories. Among experimental studies one can refer to Popov et al. [15], Schneider et al. [20], Takanashi and Ohi [21] etc. Theoretical works are those of Schneider et al. [16], Osman et al. [13] , Lee [22], Park and Paulay [14], Goel and Itani [17], Bondy [18]. The differences in seismic behaviour of systems designed by either the strong columns-weak beams (SCWB) philosophy or the weak columns-strong beams philosophy are noteworthy. These studies and those of Goel and Leelataviwat [23] showed that the moment resisting frames designed with elastic methods using equivalent static forces, may undergo large and unaccepted deformations and consequently many plastic hinges form dispersedly in different parts of the frame. A combination of ductility, shortcomings in junctions, and using unrealistic approaches in design, are the main reasons for the weak performance of SMRFs in recent earthquakes. In this study, the latter case, namely the effect of unrealistic design approaches is investigated and a more rational method based on the classic principles of plastic analysis and design combined with the principle of conservation of energy is presented. The proposed design approach leads not only to a better seismic performance but also to a better distribution of material in the vicinity of the structure.

4. Method of Combination of Basic Mechanisms

The method of combination of basic mechanisms (MCBM) was suggested by Neal and Symonds [24] and it is based on using independent equilibrium equations for a structure to derive other equilibrium equations. Using the principle of virtual work, one can write an equation of equilibrium corresponding to a collapse mechanism, thus, using independent collapse mechanisms known as *basic mechanisms*, it is possible to derive other collapse mechanisms and the corresponding collapse load factor of the structure. If a frame is loaded at nodal points, plastic hinges will only be formed on nodal points, i.e. on the member intersections, fixed supports and under point loads, because the moment diagram is linear between the end nodes of all the elements. If a frame has N critical sections and X degrees of static indeterminacy, there will be N–X independent collapse mechanisms. Other collapse mechanisms can be formed using these independent mechanisms.

Even though the selection of independent mechanisms is arbitrary, one can select those mechanisms which make calculations easier and faster. As an example, the mechanisms can be selected such that any pair among them does not pose any interaction. In this context, interaction is considered the case when two plastic hinges from two different collapse mechanisms are added together. This interaction might lead to a cancellation of that hinge or an increase in the angle of plastic rotation. It should be kept in mind that the former case is actually desirable in the MCBM. A set of basic mechanisms without any interaction, will be referred to as an *orthogonal basis* for the space of mechanisms. Figure 2 illustrates the orthogonal basic mechanisms of a planar frame. In addition to the basic mechanisms presented in Figure 2, there are some joint mechanisms which their corresponding equilibrium equations show the equilibrium of moments in that joint. Whereas the MCBM is considered as an unsafe method of plastic analysis, the mechanism with the least value of collapse load factor is closer to the exact answer. If all combinations can be considered and the minimum load factor is calculated, then the structure will collapse under this load factor. This load factor is the exact collapse load factor of the structure.

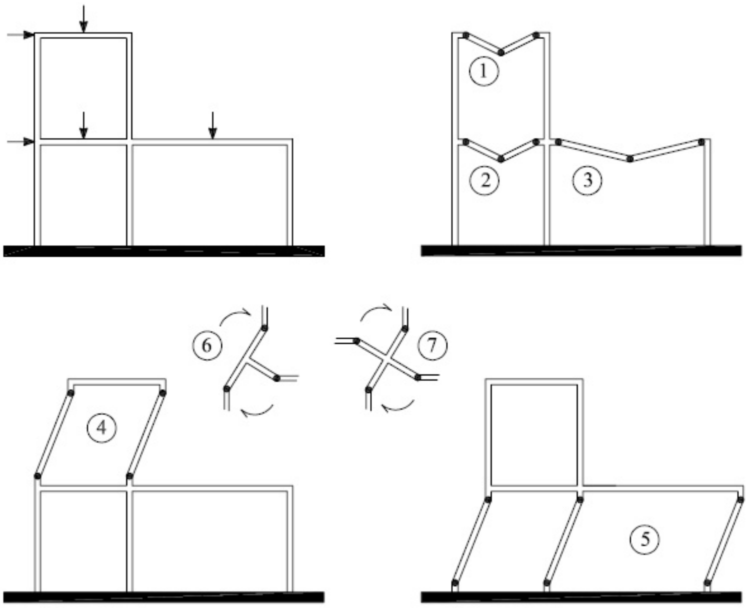

Figure. 2. A simple frame and its independent mechanisms

In the present work, the collapse load factors of all mechanisms will be adjusted such that the governing mechanism becomes a desirable one. Genetic algorithm is used as a means to search for structural member cross sections such that these members can dictate designer's desirable collapse mechanism to the structure.

5. Genetic Algorithms as Means of Search

Genetic algorithms can be considered as computer search methods based on optimization algorithms. The structure of genes and chromosomes were initially suggested by Holland, and developed and extended by some of his students, Goldberg and Ann Arbor. These algorithms simulate natural genetics mechanism for synthetic systems based on operators that are duplicates of natural ones. In the last decade, GA is used in optimum structural design. One of the first applications was the weight minimization of a 10 bar truss by Goldberg and Samtani [25]. Hajela [26], Saka [27], Adeli [28], and Kaveh and Kalatjari [29], among many others, used genetic search in design of various structures in which the search space was non-convex or discrete. Rao et al.[30] selected locations of discrete adopters in active control of structures by GA. Kaveh and Khanlari applied GA to calculate the collapse load factor of planar frames [31].

GA is founded on random process. Basically for using GA the followings should be specified:

1. Objective or cost function,

2. Search space,
3. Genetic operators.

Mating is the first operation during which some of the chromosomes are selected from population as parents. After mating, a population of chromosomes of higher fitness is generated. In fact the mating operator selects a set of the best strings but does not develop new strings, thus crossover operator is applied to the selected population to generate better strings. The goal of crossover application is to move in variables space and to conserve useful information which is hidden in strings. Crossover is a compound operator which includes three operations. First, a selection operator chooses a pair of chromosomes, randomly. Second, a random crossover site is determined and finally, values of two chromosomes in accordance to the selected crossover site are moved. After crossover operation, mutation is performed. Bit mutation includes changing 0 to 1 and 1 to 0 based on a low probability.

In most of the existing references, binary encoding is used in GA. In scientific problems the variables are often continuous and changing them into 0 and 1 is not suitable; since this leads to extensively large size of chromosomes and population. Using continuous or real variables is another approach in genetic algorithms. In this approach, design variables do not change into binary codes and are used in their original forms. Because of the following reasons, real GA is used in this paper:

- Higher speed, because of avoiding binary encoding and decoding.
- Better efficiency in computer memory occupation.
- Higher precision, because of the continuity of the search space.
- Possibility of using different techniques for defining crossover and mutation operators as well as miscellaneous and heuristic operators, proper for each problem in hand.

In this paper, arriving at a suitable collapse mechanism under seismic loading is considered as the objective of the optimum design. Here a real GA is employed to help designer to achieve a desirable collapse mechanism.

6. Optimum and Objective-Based Design of SMRFs

The first step in the design of seismic resistant structures is the calculation of conventional seismic forces determined by building codes. In the classic approach, assuming elastic behaviour, the lateral loads corresponding to the maximum expected seismic forces are determined. Then using a suitable reducing factors which are essentially dependent on structural system behaviour, lateral forces are decreased. In the design process first it is tried to provide sufficient strength and then lateral deformations are confined to permissible values. Various studies have revealed that there is not enough consistency between the proposed code forces and the permissible maximum displacements [32-33]. Most of the problems involved in accordance to SMRFs can be related to two factors. The first factor corresponds to the non-consistency between strength and lateral displacements (lateral stiffness) which is imposed by codes [33]. Most of SMRFs are designed to satisfy code requirements, free of increasing the ratio of the beam to column size. Therefore, the inelastic behaviour tends to occur in columns. The second factor corresponds to the inability of the elastic design method to capture the distribution of internal forces in the inelastic stage. Combination of these two factors leads to the formation of undesirable yield mechanisms. It is clear that new methods to design moment resistant frames should be developed in such a way that the level of force and drift requirements are compatible and the plastic distribution of internal forces is explicitly recognizable. According to these remarks, Leelataviwat et al. [32] proposed an energy-based method that results in the necessary compatibility between the magnitudes of seismic forces and desirable mechanism displacements.

6.1. Base shear calculation

The final relation for calculating the seismic coefficient using the above mentioned method is [32]:

$$\frac{V}{W} = \frac{-\alpha + \sqrt{\alpha^2 + 4a^2}}{2} \tag{1}$$

in which V is the base shear, W is total weight of the structure and α is calculated using the following relation:

$$\alpha = \left(\frac{\sum_{i=1}^{n} w_i h_i^{\,2}}{\sum_{i=1}^{n} w_i h_i} \right) \frac{\theta_p 8\pi^2}{T^2 g} \tag{2}$$

In this relation, w_i is the ith storey weight and T is the fundamental natural period of the structure. Other parameters are shown in Figure. 3. The value of a, using UBC 94 [34], can be calculated by the following formula:

$$a = ZIC \tag{3}$$

In which, Z is the seismic zone factor, I is the importance factor and C is the elastic seismic coefficient. This coefficient in UBC 94 is obtained by:

$$C = \frac{1.25S}{T^{2/3}} \le 2.75 \tag{4}$$

In this relation, the coefficient S represents the soil profile of the construction site and has a magnitude between 1 and 2.

6.2. Plastic design of frames using GA

The following approach is suggested to achieve the arbitrarily chosen mechanism which is a search process for structural member cross sections in a continuous space:

Since there are $2^{N-X}-1$ possible combinations for basic mechanisms, there will be the same number of load factors which only one of them corresponds to the desirable mechanism. If desirable mechanism load factor is shown by λ_{des}, then in the final design, the following conditions should be satisfied:

$$(S.F.)\lambda_{des} < \lambda_i \qquad i = 1,2,...,2^{N-X}-2 \tag{5}$$

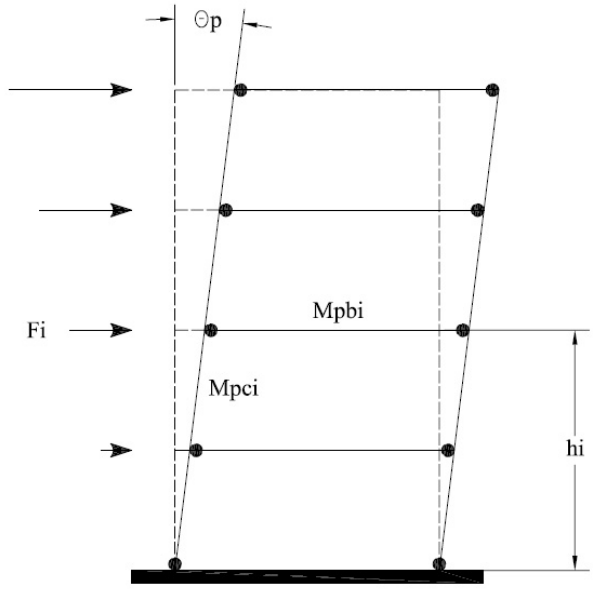

Figure 3. An arbitrarily chosen mechanism

In which S.F. is the safety factor, and λ_i is the ith load factor, which belongs to an undesirable mechanism. If all of the above conditions are satisfied, then the structure will collapse with the pre-determined mechanism. The values of λ_i is computed using virtual work equation.

Here the penalty function concept is applied to the definition of the fitness function. During solution if some of the constraints are violated, based on the quantity of violation, some penalty is imposed. When this penalty value equals to zero, the problem leads to an acceptable answer and the algorithm terminates. Thus, here the total amount of penalties which should be minimized is taken as the fitness function. In this problem there are two kinds of constraints: the first type of constraints are in inequality forms, which help always undesirable mechanism load factor to be greater than the desired mechanism load factor. Denoting the penalty function corresponding to these constraints by f, for each chromosome we have

$$f = \sum_{i=1}^{2^{N-X}-2} \alpha_i \left[(S.F.)\lambda_{des} - \lambda_i\right]^p \tag{6}$$

α_i is the penalty coefficient and is computed from the following relation:

$$\alpha_i = \begin{cases} 1 & \text{if} \quad \left[(S.F.)\lambda_{des} - \lambda_i\right] \geq 0 \\ 0 & \text{if} \quad \left[(S.F.)\lambda_{des} - \lambda_i\right] < 0 \end{cases} \tag{7}$$

p is an even number which is adjusted according to the problem, and can be determined dynamically, i.e. it is based on the amount of constraint violation.

The second kind of constrains is in the form of equality, which necessitates the collapse load factor to become equal to the desired value. Denoting the penalty function of this constraint by g, we have:

$$g = \sum_{i=1}^{2^{N-X}-2} \beta_i \left[(S.F.)\lambda_{des} - 1\right]^q \tag{8}$$

where β_i is the penalty coefficient which is defined similar to α_i:

$$\beta_i = \begin{cases} 1 & \text{if} \quad \left[(S.F.)\lambda_{des} - 1\right] \geq 0 \\ 0 & \text{if} \quad \left[(S.F.)\lambda_{des} - 1\right] < 0 \end{cases} \tag{9}$$

q is similar to p. One may use structural weight function represented by h in here, to reduce the weight of the structure simultaneously. Now the fitness function, M, is expressed as follows:

$$F = \frac{1}{1 + w_f f} + \frac{1}{1 + w_g g} + \frac{1}{1 + w_h h} \tag{10}$$

w factors correspond to different terms of the fitness function. It should be noted that here there is a pareto problem, and by assigning different values of w factors, one can obtain various answers.

After problem definition it should be encoded. In plastic design of structures, plastic moments of beams and columns cross sections are the main variables. As mentioned before, in this paper real design variables are used, to avoid encoding these into a binary system. It should be noted that due to the use of continuous variables, there is not a specified relationship between plastic moments and other mechanical specifications of cross sections. For instance one does not need to know the amount of moment of inertia or the cross sectional area for a specific M_p value. In this study by interpolation and extrapolation between standard European IPE sections properties, some formulas are derived and the rational section properties corresponding to continuous plastic moment values are computed.

6.3. Assumptions

For all the problems studied in this paper, the following assumptions are made:

- All of the frames have rigid joints, and storey heights and bay lengths are 3m and 4.5m, respectively.
- The only considered load is a combined load as 1.2DL+EQ.
- 1.2DL=16.7kN/m.
- Distributed loads are changed into equivalent concentrated loads. Baker and Heyman [35] showed that this is a good estimation in plastic analysis of steel frames.
- The effect of axial force are included. An interaction relation suggested by Beedle is used [36].
- The importance factor is taken as 1, the zone factor is considered as 0.4, and the soil profile coefficient is chosen as 1.
- The yield stress of the steel in bending is assumed to be 235.4 MPa and the effect of strain hardening is neglected.

6.4. Parametric studies and examples

The parameters of GA should be adjusted for each problem properly to cause convergence to acceptable answers as well as to raise the convergence rate. Using an improper selection of GA parameters, the search space will not be explored completely or the algorithm will be trapped in undesirable points. In this paper various parametric studies on some structures are performed in order to investigate the effect of different parameters on the quality of answers and the convergence rate of GA. In these studies optimum values for these parameters are selected such that different aspects of computing are accounted. In accordance to multiplicity of these parameters and their interaction, not all the parameters are changed simultaneously. As an example, the population size is kept constant; crossover and mutation rates are determined in accordance to this fixed population size. In this study, the population size is taken as 20. Initially a simple three-storey frame is investigated, Figure 4.

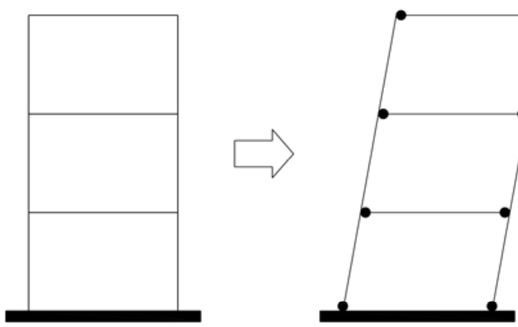

Figure 4. A three-storey frame and its collapse mode

Primarily $\theta_p = 0.01$ is assumed and the problem is solved using different parameters. After many tries, the provided software could not converge to an acceptable answer, and some of the constraints were violated. After investigations, it became clear that the algorithm converged to other mechanisms which were undesirable. These mechanisms were similar to the desirable one, with the difference of local beam mechanisms being formed in some stories. For further investigations, the equations corresponding to one desirable mechanism and an undesirable mechanism are as follows:

$$\lambda_{des}(H_1 + 2H_2 + 3H_3)h = (2M_{pc1} + 2M_{pb1} + 2M_{pb2} + 2M_{pb3})$$
$$\lambda\left[(H_1 + 2H_2 + 3H_3)h + V_3\frac{l}{2}\right] = (2M_{pc1} + 2M_{pb1} + 2M_{pb2} + 4M_{pb3}) \quad (11)$$

In the above relation, H_1, H_2 and H_3 are effective horizontal forces of stories, V_3 is the vertical (gravitational) force of the 3rd story and l is the span length of the frame. In Figure 5, the mechanisms corresponding to the above equations are illustrated. In order to achieve the ideal collapse mechanism, one should have:

$$\lambda > \lambda_{des} = 1$$

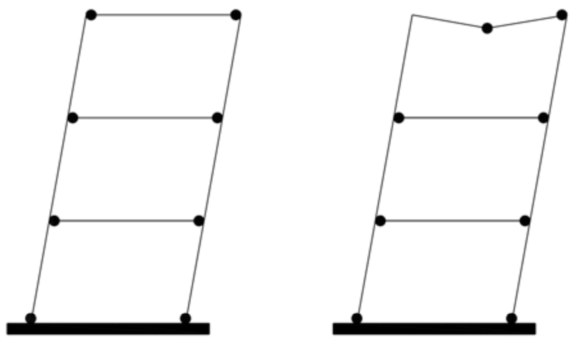

Figure 5. Mechanisms corresponding to desirable and undesirable mechanisms

or

$$\frac{(2M_{pc1} + 2M_{pb1} + 2M_{pb2} + 4M_{pb3})}{(H_1 + 2H_2 + 3H_3)h + V_3\frac{l}{2}} > \frac{(2M_{pc1} + 2M_{pb1} + 2M_{pb2} + 2M_{pb3})}{(H_1 + 2H_2 + 3H_3)h} = 1$$

In order to satisfy the above equation, it is necessary to have

$$M_{pb3} > V_3\frac{l}{4}$$

After the substitution of V_3 and l values in the above condition, the minimum value of M_{pb3} to prevent the formation of undesirable mechanism is obtained as

$$M_{pb3} > 506.25$$

Similar inequalities can be derived using the above mentioned procedure:

$$M_{pb1} > 506.25$$
$$M_{pb2} > 506.25$$

By substituting these minimum values in the main relationship corresponding to the desirable collapse mechanism, we obtain

$$2M_{pc1} + 2(506.25) + 2(506.25) + 2(506.25) = [0.91 + (2)(1.82) + (3)(2.73)](300)$$

from which M_{pc1} becomes 38.44 kN.m.

In a real and practical design, normally the dimensions of columns are decreased from first stories to top stories and if with this cross section soft storey does not form in the first storey, it will be formed in the other stories. If cross section of the beams are reduced and considered less than those determined in the above, and instead columns are strengthened, then local beam mechanisms will be formed. These results in the violation of some of the constraints and make it impossible to achieve the desired mechanism. In the other words, theoretically the structure under the applied loads will not be able to present sufficient ductility within the allowable plastic drift range. In order to solve this problem, mid-span of the beams can be reinforced. The magnitudes of the M_p at mid-spans are increased to 1.25 times by applying the Reinforcement factor (R.F.) equal to 1. The results are presented in Table 1. It should be noted that here the plastic moment of a beam is related to those of its ends.

Table 1. Some of the results obtained for $\theta_p = 0.01$

Design	**Plastic**	**Mp of Beams (kN.m)**			**Mp of Columns (kN.m)**			**Weight**
No	Drift	B1	B2	B3	C1	C2	C3	(kg)
1	0.01	54	44	44	62	58	30	3.22E+05
2	0.01	52	38	38	60	58	47	3.33E+05
3	0.01	48	46	43	66	52	22	3.08E+05
4	0.01	56	40	40	56	37	33	2.96E+05
5	0.01	45	41	38	68	68	33	3.34E+05

In order to perform parametric studies, the values of crossover and mutation rates are increased discretely and in a step by step manner. The step size for crossover and mutation rates are taken as 0.1 and 0.02, respectively. In order to analyze the obtained answers, the results are represented graphically in Figure 6.

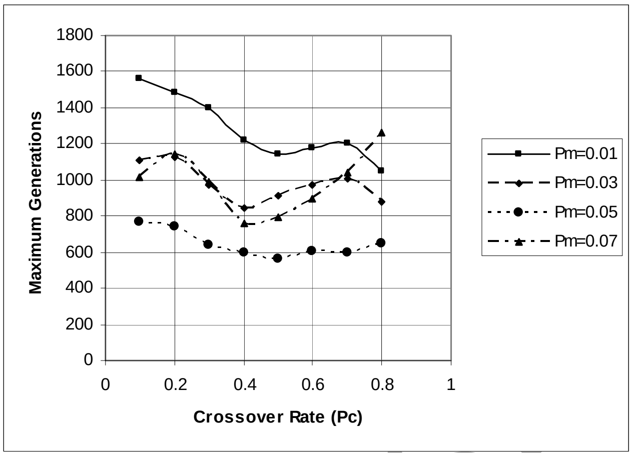

Figure 6. Results of using different GA parameters

In Figure 6, variation of the average number of generations versus crossover rate is illustrated for 4 different values of mutation rates. It is easily concluded that for different values of mutation rates, crossover rate value between 0.3 and 0.6 are resulted in least generations. Similarly, for various values of mutation rates, the best and the worst answers are obtained for 0.05 and 0.01, respectively. The bowl shape of these diagrams is interpreted as follows:

For small values of crossover rate, exploration power of the algorithm and its convergence rate is reduced. For higher crossover values, due to the high turbulence caused in the generations, convergence opportunities are decreased and again the convergence rate is increased. On the other hand, this is also true for the mutation rate. The best answers in this Figure are for a mutation rate of 0.05. For mutation rate of 0.01 convergence rate was slow and for 0.03 and 0.07 there was a similar condition and a bowl shape is produced. Of course it is obvious that the depth of this bowl is more sensitive to mutation rate in comparison to the crossover rate.

In Figure 7 another aspect of this parametric study is considered. Here the average elapsed time for each GA loop is illustrated. The parameters are taken the same as the previous diagram. This diagram has ascending behaviour for the increment of crossover rate. Because of high diversion of populations in each generation, initially computing time is high, but gradually and after some generations some convergence is developed and naturally computing time is decreased. Variation of the elapsed times during a complete GA procedure is illustrated schematically in Figure 8. The average time shown in this Figure, were used in previous diagrams. Most of the time in each loop, is consumed for populations in which crossover or mutation occurs. As the considered crossover rate is much greater than the mutation rate, most of the computation time is used for crossover operator. Now one can explain the descending behaviour of the diagram easily: the more crossover rate, the more computation time.

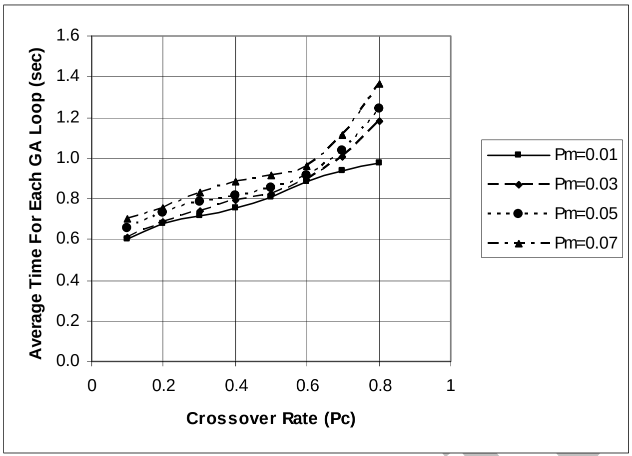

Figure 7. Average elapsed time for each GA loop verses different crossover rate

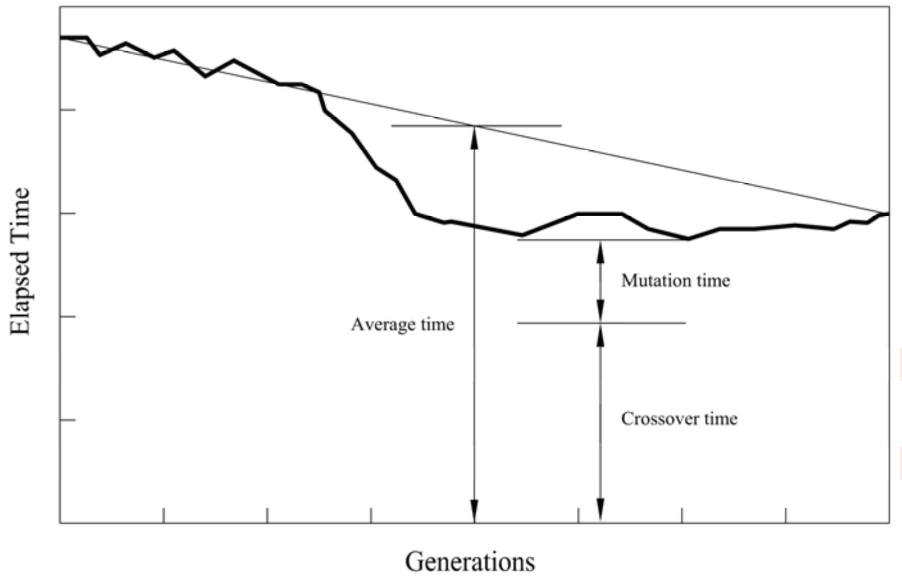

Figure 8. Schematic representation of the variation of the computational time for different generations

Figure 9 shows the total time elapsed for a complete GA process. The values on this diagram are indeed multiplication of average generation numbers and average elapsed time for each loop. This diagram has also a nearly bowl shape. Because for smaller values of the crossover rate, in spite of smaller computational time for each loop, the convergence rate is slow, therefore the total computational time increases. For greater values of crossover rate, computational time is high and convergence rate is low. Thus the right hand branch of the diagram will be ascending. For mean values of crossover rate, both loop computational time and convergence rate have average values, therefore mid-part of the diagram has smaller quantities.

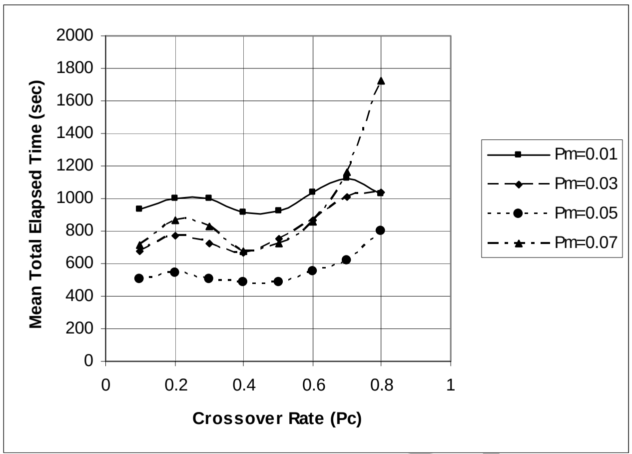

Figure 9. Total time elapsed for a complete GA process

Naturally there exists much dispersion in evolutionary algorithm solved problems, thus the results presented in this paper are indeed averaged from many solved problems. The bar diagram in Figure. 10 shows these dispersions qualitatively. In this chart, the highest bar corresponds to the highest dispersion, and the lowest bar corresponds to the lowest dispersion and the mid height bar is related to an average dispersion. Optimum parameters not only should represent the best answers and lead to the highest convergence rate but also must have the least amount of dispersion. As it is apparent from this Figure, the values 0.05 and 0.07 for mutation rates have the best performances. It seems that the value of 0.07 has more stability in comparison to 0.05. The value of 0.05 has a satisfactory performance only in the middle of the chart. It is inferred from results that the increment of mutation rate leads to the increment of stability. On the other hand, considering the previous analysis on the other aspects of this study which were illustrated in Figures. 6 to 8, the increment of mutation rate results in higher computational time, raising the maximum number of generations. Therefore, it seems that the parameters should be selected in the process of a multi aspects optimization. Considering the previous issues, the selection of 0.05 for mutation rate for crossover rate between 0.3 and 0.6 seems to be quite suitable.

Now the problem with $\theta_p = 0.02$ is studied. Concerning the increment in objective plastic drift, lateral loads are decreased. In accordance to the previous experience, it was predicted that GA can not achieve a desirable fitness and this happened. Again, similar to the previous condition, mid-span of beams were reinforced. For the values of 1.25, 1.50 and 1.75 of R.F., the algorithm could not converge but it converged for R.F.=2.00. It should be noted that for this case random parameters were checked and the previous results were obtained.

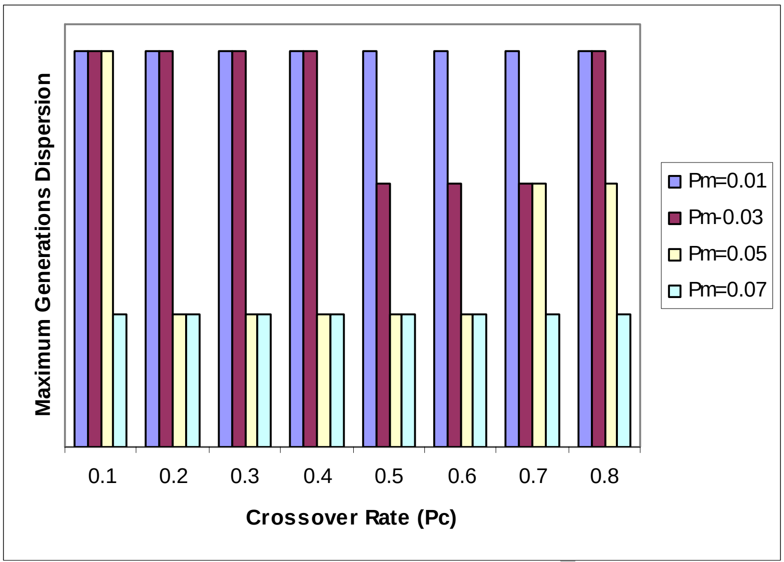

Figure 10. Qualitative dispersions for different crossover rate

Using similar procedures, some other examples are investigated and similar results are obtained. The only difference was in the computational time and the maximum number of GA loops, however, values of GA parameters had no major variations. These examples together with the average time consumed for each loop for 0.5 and 0.05 values of crossover and mutation rates, respectively, are illustrated in Figures 11 and 12.

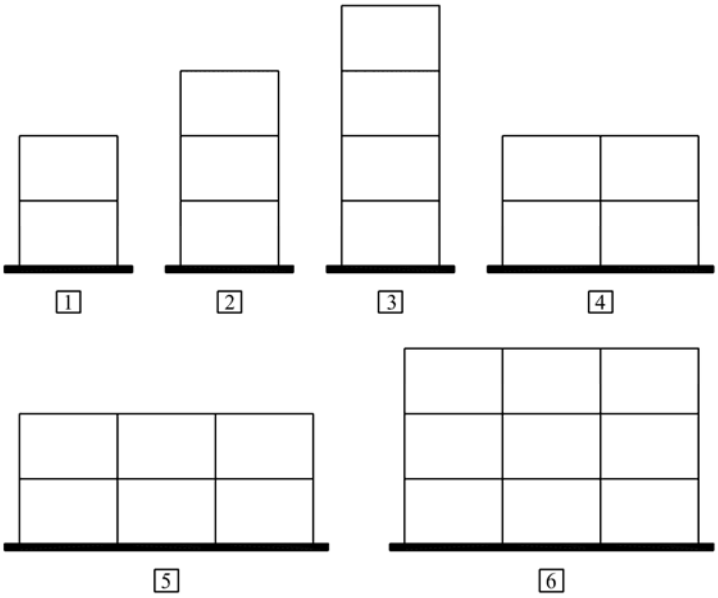

Figure 11. Six frame examples

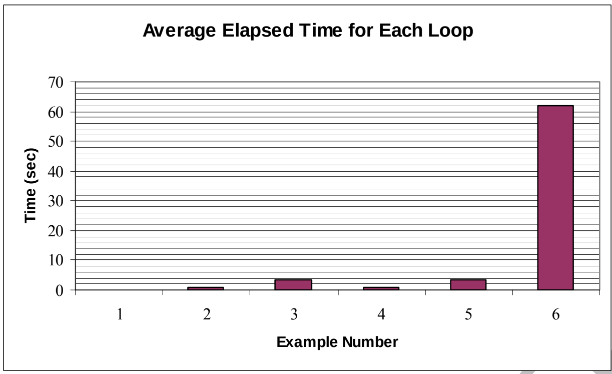

Figure 12. Average time consumed for each loop in six examples

It is necessary to add some remarks about Table 2. First, the presented R.F. value for each case has commenced from its least possible value, i.e. it is impossible for GA to converge for small values of R.F.. It can be observed that most of the answers are obtained for R.F.>1.0. Secondly, Example 4 shows that, three kinds of results are obtained, which are schematically illustrated in Figure 13. Similar answers are observed in the subsequent examples.

Table 2. Some of the results of design for the investigated examples

Example	Mp	Element ID						R.F.	θ_p	Example
Number	(kN.m)	1	2	3	4	5	6			Number
1	Mpb	40	37					1.25	0.01	1
	Mpc	40	25							
1	Mpb	28	26					1.50	0.01	1
	Mpc	48	44							
2	Mpb	54	44	44				1.25	0.01	2
	Mpc	62	58	30						
2	Mpb	52	45	27				1.50	0.01	2
	Mpc	60	47	30						
2	Mpb	27	24	17				2.00	0.02	2
	Mpc	33	22	19						
3	Mpb	65	59	55	52			1.25	0.01	3
	Mpc	85	79	76	20					
4	Mpb	54	50					1.00	0.01	4
	Mpc	56	37	38	24					
4	Mpb	50	48					1.00	0.01	4
	Mpc	31	69	20	53					
4	Mpb	49	47					1.00	0.01	4
	Mpc	43	61	30	25					
5	Mpb	57	53	45	43			1.25	0.01	5
	Mpc	35	60	29	35					
6	Mpb	69	73	68	70	62	19	1.50	0.01	6
	Mpc	88	60	69	46	62	28			

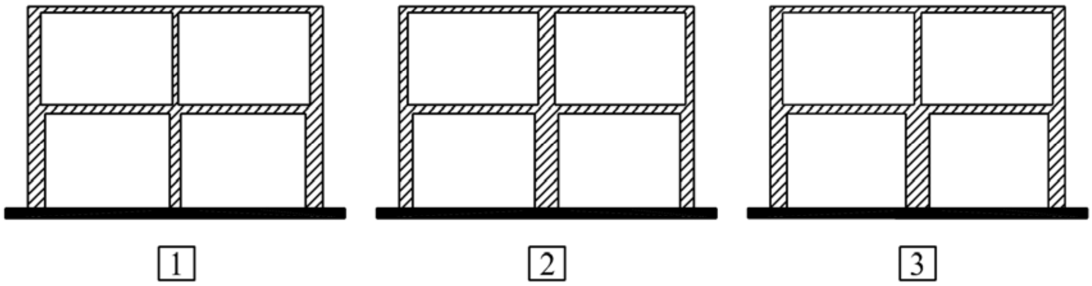

Figure 13. Frames with different proportioning of structural members

All of these answers satisfy the conditions of the problem, however, these are different in some senses. In all of these results, some of the load factors correspond to other combinations of mechanisms which are close to the desirable mechanism load factor. There is fear that due to the uncertainties in fabrication and construction and even in the determination of seismic forces, undesirable mechanisms govern the problem. From this point of view, the first mechanism in Figure. 13 which has the least number of undesirable close mechanisms, has a better condition compared to the other two illustrated mechanisms. Some suggestions will be provided on this problem at the conclusion section of this paper

6.5. Application in reinforcement and rehabilitation

The suggested idea in this paper can be used for improving the behaviour and the load bearing capacity of the existing weak structures. Here, an example is presented. It is assumed that there exists a structure as specified in Figure. 14 and Table 3. It is desirable to evaluate seismic behaviour of this structure under code forces and if necessary to rehabilitate it in an optimum manner. Plastic analysis of the mentioned structure using provided software revealed that not only the structure has a soft storey collapse mechanism but it will collapse under the applied loads with a load factor of 0.68 which is less than unity. Thus the structure is weak and needs to be rehabilitated. Using R.F.=1.5, the structure is reinforced and redesigned. Using optimum values of 0.5 and 0.05 for crossover and mutation rates, respectively, GA converged within 120 loops with its convergence history depicted in Figure 15. In Figure 16 the yield mechanism after rehabilitations, and in Table 4 the new M_p values are shown. As mentioned before the mid-span values of M_p are 1.5 times greater than beams ends.

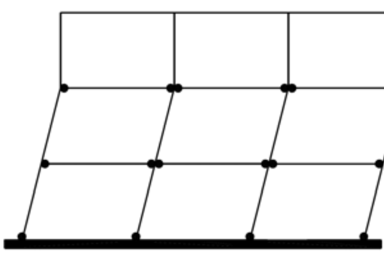

Figure 14. The yield mechanism before rehabilitations

Table 3. The structural members

	beam			column			
story	**1**	**2**	**3**	**1**	**2**	**3**	**4**
1	IPE 180	IPE 180	IPE 180	IPE 200	IPE 200	IPE 200	IPE 200
2	IPE 180	IPE 180	IPE 180	IPE 180	IPE 180	IPE 180	IPE 180
3	IPE 160	IPE 160	IPE 160	IPE 180	IPE 180	IPE 180	IPE 180

before rehabiltation (λ=0.68)

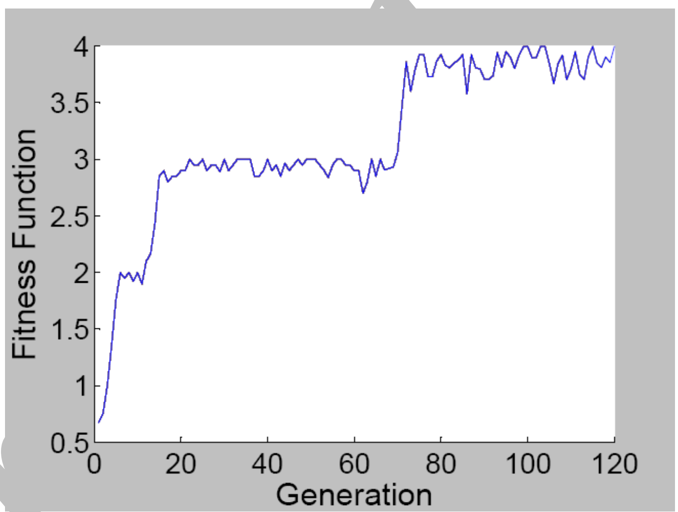

Figure 15. Convergence history of GA for the 3-span 3-storey frame

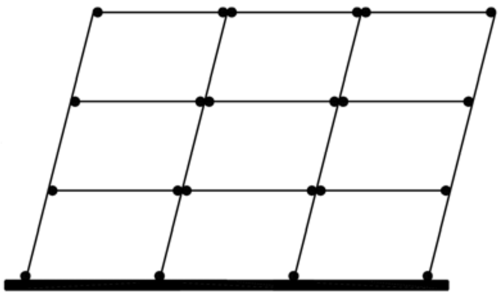

Figure 16. The yield mechanism after rehabilitations

6.6. Promoting the efficiency of GA

In order to increase the efficiency of GA, some suggestions are also made. Since in the plastic method the form and type of the search space for solving the plastic design are convex, therefore the upper limit for cross sections are selected proportional to its dimensions. For this purpose, employing the equation of external virtual work, the total amount of the work which should be done by seismic forces on the desirable mechanism, is calculated as

$$W_{int} = (H_1 + 2H_2 + 3H_3)h\theta_p \tag{12}$$

in which, W_{int} is the total internal virtual work. Then considering the internal work which should be done in the desirable mechanism, estimation is made for the limits of cross sections M_p. Now using this estimation, upper limits can be determined.

Controlling the mutation rate is another approach for making the convergence faster. Considering the parametric studies explained in the previous sections, it can be concluded that higher the mutation rate is selected, more turbulence will be produced in the generations. Consequently the turbulence makes the algorithm convergence slower. Thus in order to accelerate the GA convergence, the mutation rate should be adjusted such that in the vicinity corresponding to the limit state value of the fitness function, the mutation rate is decreases. This will prevents the turbulent in generations in the process of complete convergence.

Another point is in relation with using the general variations of member cross sections trend in the stories. In the structural frames column dimensions usually decreases from down to up. This rule is true for the beams of each bay. Thus remembering these practical rules, some conditions are added to the fitness function to take care of this general rule.

In order to conserve the best populations of each generation, the elitism concept is proposed. Elitism can have a great share in raising the efficiency of GA, however this is not always true in this problem. Since there may exist many designs not corresponding to undesirable mechanisms with fitness functions close to the limit state fitness value. Consequently, selection of elite population in each generation and keeping it for the next generation can lead to premature convergence to improper answers and under such condition even mutation operator can hardly distance the GA from vicinity of these undesirable answers. However this needs more time and postpones the convergence. Therefore it was decided not to apply the concept of elitism in this case.

7. Concluding Remarks

In spite of some preliminary beliefs, the design of SMRFs according to weak beam-strong column rule is not simple. In most current methods based on the elastic design of structures, the structure is not often optimally designed. Incidentally, a desirable mechanism is rarely arrived at. In conventional methods one can merely reinforce the columns such that the collapse mechanism passes through special paths causing the formation of plastic hinges only in the beams. Finally, if the load is increased, other mechanisms such as beam mechanisms or a soft storey will most likely be formed, because as shown in this research, the space with no constraint violation is too narrow and irregular. Since this will lead to a sensitive design, therefore the final design should be constructed with more accuracy.

Another important conclusion which can be derived is that the design of SMRF for an arbitrary collapse mechanism and a value of ductility via members of constant cross sections is not always feasible. In order to have a structure with sufficient ductility, at least theoretically, variable cross sections should be used for beams.

If performance-based design of structures using current approaches can not control the collapse mechanism of the structure, the designer will not be able to achieve a number of design goals, thus control of the collapse mechanism under applied loads should be a major consideration during the design process.

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