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Robust Design of Reinforced Concrete Moment-resisting Frames

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ABSTRACT

Reinforced concrete moment resisting frame (RCMRF) is one of the most popular structural systems. Conventionally, buildings with RCMRF systems are designed to satisfy the relative displacement, resistance, and flexibility requirements defined by the design codes. Structural design codes have given different ranges of design parameters that the designers and engineers must consider in the design process of structures and the values selected for these parameters affect the seismic behaviour of the structures. However, performance assessment of the RCMRF under the earthquake loading to limit the probable levels of damage has a complicated and difficult procedure that is time-consuming for designing of ordinary buildings. In this study, to prevent this time-consuming process, tighter ranges for design parameters have been attempted to improve the seismic performance of the RCMRFs. In this regard, databases of RCMFs were created for different ranges of design parameters. The Particle Swarm Optimization (PSO) algorithm is used to create these databases and RCMRFs are optimally designed according to ACI 318-14 code. Then, nonlinear time history analysis according to ASCE/SEI 7-16 code was performed on the RCMRFs in each one of the databases and the statistical analysis of local and global results acquired from the nonlinear time history analysis is carried out. Finally, tighter ranges of design parameters have been determined to achieve more robust structures without involvement in time-consuming processes.

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1. INTRODUCTION

Recent developments in construction technologies have made the design of more complex structures easier which are susceptible to undesirable effects of severe events such as an earthquake. It is difficult to prevent the collapse of structures under such events; nevertheless, the consequences of failure can be reduced significantly in the structures with adequate robustness. It seems that structural robustness can be suggested as a novel key concept in the design of concrete structures; however, quantification and methods of robustness assessment have not been sufficiently integrated yet. This study attempts to facilitate the design of robust structures in the medium-rise reinforced concrete (RC) buildings, which make up a large part of buildings, by providing simple approaches that are applicable and known to the structural engineers.

Structural designers have different strategies in selecting design parameters for RC frames. This issue drastically affects the performance of structures under earthquake and changes the robustness of the structures. This study uses Particle Swarm Optimization (PSO) algorithm as a well-known algorithm for structural design to avoid such issues and evaluate the impact of the selected parameters on robustness of the structures.

In specified ranges for various parameters, the structures designed by this algorithm are analyzed under the selected earthquakes using Non-Linear Time-history Analysis (NLTHA). The impact of design parameters variation on the seismic performance of the structures were assessed by employing statistical methods. Therefore, suitable parameters range can be determined to increase the structural robustness subject to earthquake loading.

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2. DESIGN ALGORITHM

Based on the reasons mentioned above, the design optimization algorithm selected in the present study is PSO algorithm which is one of the best metaheuristics and has been widely used in the past years.

2. 1. Sections Database A variety of different reinforcement patterns and sections can be used for the beams and columns in the RC frames. In this section, two main databases are developed for the sections of beams and columns to reduce the complexity of the optimization process. To create these databases, the provisions of the design code and some practical requirements are followed. Most concrete columns and beams usually have square and rectangular sections, which their length to width ratios vary between 1.1 and 2. The increment of dimensions of the sections is usually 5 cm. The size of the steel reinforcement is F18, F20, and F22 for beams and F20, F22, and F25 for columns. The strength of concrete is considered 280 kg/cm^2, longitudinal steel reinforcements, 4200 kg/cm^2, and shear reinforcements, 3000 kg/cm^2.

ACI 318-14 [1] code applies certain requirements for sections. These requirements include the minimum and maximum steel reinforcement ratio, the minimum concrete cover thickness of 4 cm for the reinforced concrete members, minimum diameter of the shear steel reinforcements, and the minimum space between longitudinal reinforcements. Complying with these requirements, a large number of sections can be created for beams and columns.

It is worth to mention that the requirements specified in ACI 318-14 code related to the frames with special ductility, were taken into consideration when developing the database [1].

2. 1. 1. Beams According to the ACI 318-14 Code [1], the following requirements should be considered for the beam sections:

1. As Figure 1a shows, at least four bars should be positioned at the four corners of the section.
2. The minimum space between the longitudinal bars should be 4 cm.
3. The minimum thickness of the concrete cover should be 4 cm.
4. The diameter of the ties is F10.
5. The bar layers should be limited to two layers.
6. The upper-layer reinforcements should be located in the same position as the lower layer reinforcements, and as shown in Figure 1b, the minimum free space between two layers should be 2.5 cm.
7. If a beam section requires a larger number of bars, all these reinforcements will be added to the second layer, symmetrical to the vertical axis of the section, and exactly above the lower layer of reinforcements. If the mentioned symmetry cannot be obtained, then the symmetry is to be created by adding another reinforcing bar, as shown in Figure 1c [2]

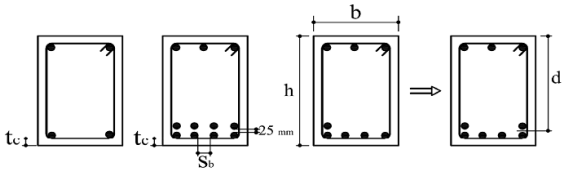
The requirements of section 10 of ACI 318-14 Code are observed for the minimum and maximum ratio of the flexural reinforcing bars [2]. Considering the mentioned provisions, 35 different dimensions of sections are created as follows:

$35 \times 40 \sim 70, 40 \times 45 \sim 75, 45 \times 50 \sim 80, 50 \times 55 \sim 85, 55 \times 60 \sim 90\ cm$

A total number of 8548 different sections are created for beams and a sample of the details is presented in Figure 2. This beam sections used in the present study.

2. 1. 2. Columns According to ACI 318-14 Code [1], the following requirements should be observed for the column sections:

A. The minimum clear spacing between the longitudinal reinforcing bars should be considered s_c=40mm.
B. The least number of reinforcing bars is four, and as shown in Figure 3a, they should be positioned at the four corners of the section.
C. The minimum concrete cover should be t_c=40mm.
D. Considering the special ductility of the frame, the diameter of ties should be F10, and their clear spacing is set at 12.5 cm.
E. As Figure 3b shows the arrangement of the bars should be symmetrical in the two opposite sides of the section.



(a) (b) (c)

Figure 1. Restrictions for placement of reinforcements in the beams: (a) at least four reinforcing bars in the corners. (b) The minimum clear spacing between the longitudinal bars in two layers. (c) Reinforcing bar placement symmetry in relation to the vertical axis of the section. [2]

Beam Number	Width (cm)	Depth (cm)	Area (cm^2)	Moment of inertia (cm^4)	Number of bars		Bar size (mm)	Factord moment resistance (kg.cm)		Cost per unit length ($)
					Center	End		Center	End	
1	35	40	1400	186666.67	2	2	18	631328.03	631328.03	127.69
2	35	40	1400	186666.67	3	2	18	928479.89	631328.03	129.49
3	35	40	1400	186666.67	4	2	18	1213290.30	631328.03	131.29
4	35	40	1400	186666.67	2	2	20	769894.37	769894.37	129.38
5	35	40	1400	186666.67	3	2	20	1126626.09	769894.37	131.60
6	35	40	1400	186666.67	4	2	20	1464547.51	769894.37	133.82
7	35	40	1400	186666.67	2	2	22	919138.98	919138.98	131.24
8	35	40	1400	186666.67	3	2	22	1337398.22	919138.98	133.93
9	35	40	1400	186666.67	4	2	22	1728117.30	919138.98	136.61
10	35	40	1400	186666.67	2	3	18	631328.03	928479.89	129.49
11	35	40	1400	186666.67	3	3	18	928479.89	928479.89	131.29

Figure 2. Beams Database

F. The area of the longitudinal reinforcing bars should be between 1 and 3% of the total cross-section [2].
G. The maximum clear spacing between the longitudinal reinforcing bars should be considered S_c=150mm.

F20, F22 and F25 reinforcing bars are used in the column sections. A database containing 150 types of square sections with 40 cm to 90 cm dimensions and 5 cm steps, has been used for the columns. The strength of each column section under the applied loads (flexural and axial) is calculated using the P-M interaction curves. Eleven-point P-M interaction linear- diagram shown in Figure 4 has been used in this study.

2. 2. Frame Analysis All design conditions and requirements must be considered for the optimal design of a frame. To do so, internal forces such as axial forces, shear forces, and bending moments of each member are needed. These structural response values are calculated to design each of the frames using Finite Element Analysis. In the present study, only bending moments have been considered for the beams to simplify the calculations, while combined axial forces and bending moments have been considered for the columns. The analysis of columns also includes checking the column slenderness. If a column is determined to be slender, the slenderness coefficient is applied. According to ACI 318-14 Code, if a column is slender, the moment amplifies.

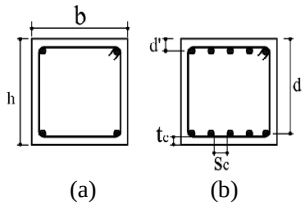


Figure 3. Limitations of reinforcing bar placement in the Columns: (a) At least four longitudinal reinforcing bars in the four corners. (b) Symmetrical patterns of bars and the bar clear spacing and cover [2]

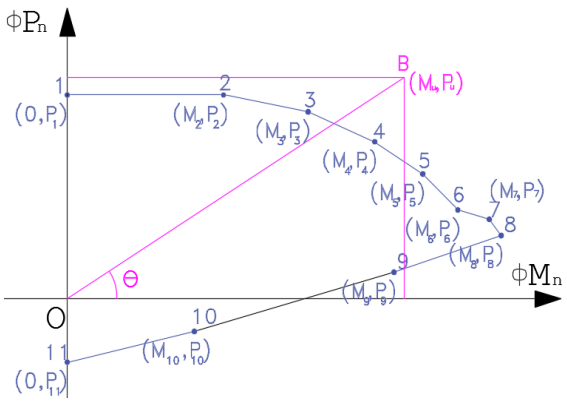


Figure 4. Eleven-point P-M interaction linear- diagram

2. 3. Geometric Constraints of the Problem In this constraint, the section dimensions and the number and size of the reinforcing bars of the upper floor column should be less than or equal to the lower floor column. The geometric structural members are shown in Figure 5.

Furthermore, in the beam-to-column connections, the beam width should be less than or equal to the width of the lower floor column, the dimensions of columns on each floor are also the same, but their reinforcing bars can be different, and the beams of lower floors have greater or equal dimensions to the beams of upper floors. These constraints are written as follows:

$$\begin{aligned} g_m^b &= \frac{b_b}{b_{cd}} - 1 \le 0, m = 1,2,\dots.nj \\ g_m^{bc} &= \frac{b_{cu}}{b_{cd}} - 1 \le 0, m = 1,2,\dots.nj \\ g_m^{bb} &= \frac{b_{bu}}{b_{bd}} - 1 \le 0, m = 1,2,\dots.nj \\ g_m^{bcie} &= \left|\frac{b_{ci}}{b_{ce}} - 1\right| \le 0, m = 1,2,\dots.nj \\ g_m^{dc} &= \frac{d_{cu}}{d_{cd}} - 1 \le 0, m = 1,2,\dots.nj \\ g_m^{bh} &= \frac{h_{bu}}{h_{bd}} - 1 \le 0, m = 1,2,\dots.nj \end{aligned} \quad (1)$$

2. 4. Design Constraints The constraints related to a force-based design should also be incorporated into the design and analysis process. For the beams, the corresponding moment demand in the middle (M_u^+) and at the ends of the member (M_{uR}^- andM_{uL}^-) should be less than the section capacity ($\emptyset M_n$). For the columns, the capacity of the column sections should also be larger than the corresponding demand. The interaction of axial force and bending moment are considered to assess the capacity of columns. Therefore, the pair of axial forces and bending moment (M_u, P_u) resulting from the imposed loads should not exceed the range of the column's interaction diagram. The section capacity and demand are calculated as follows to formulate this column constraint.

$$L_n = \sqrt{(\phi M_n)^2 + (\phi P_n)^2},\ L_u = \sqrt{(M_u)^2 + (P_u)^2} \quad (2)$$

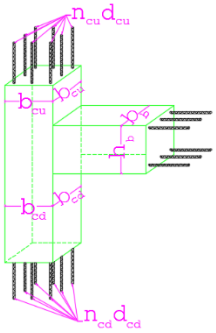


Figure 5. Geometries of structural members

Thus, if $L_u \leq L_n$, then the column section can be considered adequate. Finally, the formulation of constraints for the reinforced concrete frame can be expressed as follows:

$$g_1 = \frac{M_u^+}{\varphi M_n^+} - 1 \leq 0 \quad (3)$$

$$g_2 = |M_{uL}^-|/\varphi M_n^- - 1 \leq 0 \quad (4)$$

$$g_3 = |M_{uR}^-|/\varphi M_n^- - 1 \leq 0 \quad (5)$$

$$g_4 = \frac{L_u}{L_n} - 1 \leq 0 \quad (6)$$

2. 5. Constraints of Inter-story Drift According to 16.4.1.2 of ASCE 7-16 Code, the mean inter-story drift should not be larger than twice the values available in ASCE 7-16 Code (see Table 12.12-1). The inter-story drift should be computed as the difference of the deflections at the centres of mass at the top and bottom of the story under consideration [3].

$$\delta_x = \frac{C_d \delta_{xe}}{I_e} \quad (7)$$

C_d is the deflection amplification factor, calculated from ASCE 7-16 Code, Table 12.21, δ_{xe}, the deflection at the location required, determined by an elastic analysis, and I_e, the importance factor determined in accordance with ASCE 7-16 Code (section 11.5.1). The C_d coefficient for the reinforced concrete special moment frame is 5.5, which is extracted from ASCE 7-16 Code (Table 12.2-1) and I_e is obtained from ASCE 7-16 Code (section 11.5.1) equal to 1 [3], therefore:

$$\frac{C_d \delta_{xe}}{I_e} \leq \Delta_a \rightarrow \frac{C_d \delta_{xe}}{I_e} \leq 2 \times 0.02 \times h_{sx} \rightarrow \frac{\delta_{xe}}{h_{sx}} \leq 2 \times \frac{0.02}{5.5} = 0.00727 \quad (8)$$

h_{sx} is the story height below Level x. Ultimately, the formulation of the drift constraints for the inter-story drift of mentioned RC frames can be written as follows:

$$g_{di} = \frac{\delta_{xe}}{h_{sx} \times 0.00727} - 1 \leq 0 \quad , \quad i = 1,2, \ldots . n \quad (9)$$

2. 6. Strong Column Weak Beam (SCWB) Constraint ACI 318 adopts the strong-column/weak-beam principle by requiring that the sum of column moment strengths exceed the sum of beam moment strengths at each beam-column connection of a special moment frame. Studies conducted by Kuntz and Browning [3] and Maffei [4] have shown that the full structural mechanism can be achieved only when the column-to-beam strength ratio is relatively large (about four or more). Because this ratio is impractical in most cases, a lower strength ratio of 1.2 is adopted by ACI 318 [1]. The following inequality should be observed in all the structural joints to prevent the development of the mentioned state:

$$M_{n,top}^{col} + M_{n,bot}^{col} > \frac{6}{5}(M_n^+ + M_n^-) \quad (10)$$

In the expression above, the sides of the expression are the total plastic moment capacity for the members of the beams and columns at each structural joint.

2. 6. Optimization Formulation The formulation of optimal design in force-based design methods is as follows.

$$minimize: F(x)$$
$$subject\ to: g_i(x) < 0 \ , \ i = 1,2, \ldots ,n \ , x^L < x < x^U \quad (11)$$

In expression (7), x vector indicates the design variables, and F is the optimization objective function, which is a criterion for selecting the best designed structure. This part of the study aims to minimize the construction costs of the structure. Needless to mention, minimizing the objective function should not have an adverse impact on structural behaviour and efficiency. Therefore, the minimum point of the objective function should satisfy $g_i(x) < 0$ inequality, which is also termed as the problem constraints.

The main goal in the optimization of a RC frame is to minimize the construction cost, thus the objective function can be written as follows:

$$C = \sum_{i=1}^{nb} (C_C\, b_{b,i}\, h_{b,i} + C_S\, A_{S,b,i} + C_F\, (b_{b,i} + 2\, h_{b,i})\, L_i + \sum_{j=1}^{nc} (C_C\, b_{c,j}\, h_{c,j} + C_S\, A_{S,c,j} + 2C_F\, (b_{c,j} + h_{c,j}))H_j \quad (12)$$

In which C is the objective function, n_b the number of beams, $b_{b,i}$, $h_{b,i}$, L_i and $A_{s,b,i}$ are the width, height, length and the area of the bars in the i^{th} beam respectively, n_c is the number of columns, $b_{c,j}$,$h_{c,j}$,H_j and $A_{s,c,j}$ are the width, height, length, and the area of the bars in the j^{th} column respectively. C_C, C_S and C_F are the cost of each unit of volume of concrete, steel, and the cost of the unit of area of moulding according to American Society of Civil Engineers [5] respectively. Their values are $C_C = 105\ \$/\ m^3, C_S = 7065\ \$/\ m^3, C_F = 92\ \$/\ m^2$.

In this research, the constraints of the optimization problem have been applied using the concept of penalty function [6]. Thus, the penalty functions are written as below:

$$\Phi = F(1 + P_{beam} + P_{column}) \quad (13)$$

$$P_{beam} = r_p \sum_{i=1}^{nb} ((\max\{0, g_1\})^2 + (\max\{0, g_2\})^2 + (\max\{0, g_3\})^2)_i \quad (14)$$

$$P_{column} = r_p \sum_{j=1}^{nc} ((\max\{0, g_4\})^2 + (\max\{0, g_5\})^2 + (\max\{0, g_6\})^2 + (\max\{0, g_7\})^2)_j \quad (15)$$

In these equations, r_p is the positive parameters of the penalty function, P_{column} and P_{beam} are the penalty functions of the column and beam members [7].

3. PARTICLE SWARM OPTIMIZATION ALGORITHM

Recently a group of optimization algorithms has been created based on the simulation of the social interaction of a group of live creatures for achieving food resources. Particle Swarm Optimization (PSO) algorithm [8, 9] which was introduced by Eberhart and Kennedy [2], is an algorithm of that group. The PSO algorithm has had a very strong performance in comparison to similar algorithms and it can easily work with continuous and discrete variables and integers. In comparison to similar optimization methods, the PSO algorithm is much more effective and needs less function call to obtain better or similar results compared to other algorithms. It involves a very simple concept, and paradigms can be implemented in multiple lines of computer code, which conveniently accommodates the constraints and variables of a specific problem [10]. The formulation of this algorithm is described in literature [11-13].

4. DESIGN OF REINFORCED CONCRETE FRAMES

4. 1. Geometry of Frames For the force-based optimal design, two 6-story and 9-story, 3-bay frames were considered. In designing of these frames, the created databases for beam and column sections have been used. To create an efficient and limited search space of beams and columns, Lee's method [6] for optimization has been used. The capacity of sections obtained based on the ultimate capacity of the section by coding in MATLAB [14]. The frames being studied are the 3-bay frame from the middle axes of the 6-story and 9-story building plan, as shown in Figure 6.

4. 2. Loading the Frames In the models being studied, the lateral static loads of earthquake in the form of horizontal point load applied on the nodes of each story and the gravity loads for the dead load assumed as DL=500 kg/m^2 and LL=200 kg/m^2 for the live load.

The earthquake force was calculated according to ASCE 7-16 Code. The seismic coefficient for the 6-story building has been determined as 0.125, and 0.118 for the 9-story building, and earthquake base shear calculated and distributed along the height based on the expressions of the Code [3]. The loading combinations applied according to ACI 318-14 [1] for the assessment of demands in all the models.

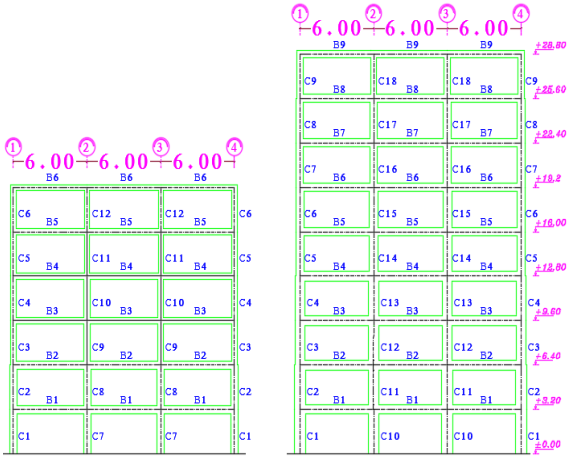


Figure 6. Typing of the selected frame for the 6-story and 9-story buildings (dimensions are in meters)

4. 3. Specifications of Materials The value of yield stress of the steel reinforcing bars and the compressive strength of the concrete in all the frames assumed as $f_c' = 280\ kg/cm^2, f_y = 4200\ kg/cm^2$.

5. ROBUSTNESS ASSESSMENT OF DESIGNS

In the real-world engineering, some uncertainties are considered during the design process. Considering the natural properties of the problem, the uncertainties would always exist, and depending on the source, the uncertainty can be reduced but cannot be avoided [15]. Uncertainty is not considered in definitive optimization. On the other hand, in the Robust Design Optimization (RDO), an additional objective function related to the random nature of problem parameters in the demand and capacity is considered. Therefore, for the optional structures in RDO, when the structural specifications or the seismic loading are considered as random variables, the aim is to minimize the initial cost and variation of the responses. A complete study of the available research papers about robust design optimization can be found in literature [16]. The RDO problem is stated as a multi- objective optimization problem as per the following description:

$$\begin{aligned} &\min_{s \in f} \quad [C_{in}(\boldsymbol{s}), COV_{EDP}(\boldsymbol{s}, \boldsymbol{x})] \\ &subject\ to: g_i(\boldsymbol{s}, \boldsymbol{x}) \geq 0\ i = 1, \dots, l \\ &s_j \in D^{n_d}\ j = 1, \dots, m \end{aligned} \tag{16}$$

where s and x represent the design and the random variables vectors, respectively. The objective functions considered are the initial construction cost, C_{in}, and the coefficient of variation of an EDP, COVEDP. The

conceptual difference between a DBO and an RDO optimisation problem, is explained schematically in Figure 7 [17].

5. 1. Assessment of the Design Fundamentals Affecting the Structure Robustness The input optimization database is reformed based on the variations of requirements in codes for different ranges of ρ of the columns (1%≤ρ<1.5%, 1.5%≤ρ<2%, 2%≤ρ<2.5% & 2.5%≤ρ≤3%) and beams (ρ_{min}<ρ≤1%, 1%<ρ≤ρ_{max}) and the best answers (optimized structures) obtained for each of these databases using the optimization algorithm. Figure 8 shows the created database for the 6-story structure with the values of ρ of columns.

5. 2. Selection of Answer Set It consists of N points from the end of the optimization convergence chart according to the initial costs, in which the points are the answers resulting from the optimization of a structure sorted by descending values of the objective function.

It is obvious that the required number of points must be selected according to the value of the acceptable error in the calculation of the standard deviation and mean, using the expressions in the statistical discussions. In this study, the Cochran formula has been used to determine sample size. This equation is as follows [18]:

$$n = ((z^2\, pq)/d^2) \,/\, ((1 + 1/N\, ((z^2\, pq)/d^2 - 1)\,)\,) \quad (17)$$

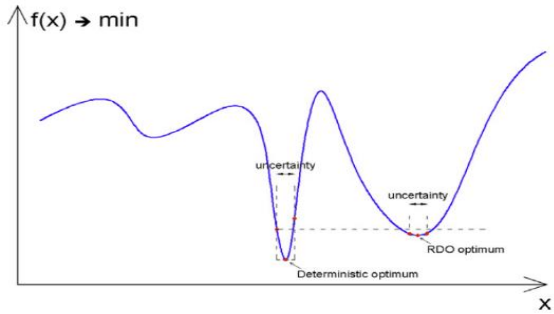


Figure 7. The concept of robust design optimisation [16]

In the equation above, n is the sample size, N, the statistical population size, z, the normal variable of the standard unit, which is equal to 1.96 at the confidence level of 95%. p is the proportion of the population that has the attribute in question and if not available, it will be considered 0.5. q is the proportion of the population that does not have the attribute in question and equals (1-p). d is the acceptable margin of error, which is usually considered 0.01 or 0.05 [18].

Based on the mentioned equation, the number of structures to be studied for the robustness is about 40, and to increase the accuracy of the results in this study, the tolerance of the objective function was limited to 5%. In each range, the most economic structures within this limit were selected and their number is larger than 40 structures. A sample of the created database is shown below.

5. 3. Non-Linear Time-History Analysis To investigate the effect of the variations in design parameters on the robustness of the structures, using the 12 accelerogram records of the surrounding area, nonlinear analysis of time history was performed for each of the selected statistical populations in section 5.2. The schematic chart of scaling the accelerograms is shown in Figure 9. These accelerograms were randomly extracted from the PEER databank and according to the method recommended in ASCE/SEI 7-16, each one was scaled to a 2% probability of exceedance in 50 years [19].

The expected yield stress of steel and strength of concrete was used in the non-linear time-history analyses according to ASCE/SEI 41-13 Code [20]; therefore, the expected yield stress of steel and strength of concrete is determined as f_y=5250 kg/cm^2, f_c'=420 kg/cm^2. Thus, the expected strength of unconfined concrete was fc'=420 kg/cm^2, the corresponding strain, 0.002, the ultimate strength, 224.0 kg/cm^2, and its corresponding strain, 0.004. The pattern suggested by Mander et al. [21] has been used in this study to define the non-linear stress-strain curve of the concrete materials in the compressive zone and in the confined

Structure #	Beams						Columns												Cost $
	B1	B2	B3	B4	B5	B6	C1	C2	C3	C4	C5	C6	C7	C8	C9	C10	C11	C12	
1	5540051018	5540051018	5540040720	5040040818	5040030620	5035020422	601222	551220	501020	501020	501020	451020	601420	551220	501020	501020	501020	451020	37884
2	5540061018	5540051018	5540051018	5040030622	5040030620	5035020422	601222	551220	501020	501020	501020	451020	601420	551220	501020	501020	501020	451020	37979
3	5540061118	5540051018	5540051018	5040030622	5040030620	5035020422	601222	551220	501020	501020	501020	451020	601420	551220	501020	501020	501020	451020	38041
4	5540071118	5540051018	5540051018	5040030622	5040030620	5035020422	601222	551220	501020	501020	501020	451020	601420	551220	501020	501020	501020	451020	38073
5	5540061118	5540061118	5540051018	5040030622	5040030620	5035020422	601222	551220	501020	501020	501020	451020	601420	551220	501020	501020	501020	451020	38106
6	5540071118	5540061118	5540051018	5040030622	5040030620	5035020422	601222	551220	501020	501020	501020	451020	601420	551220	501020	501020	501020	451020	38138
7	5540071118	5540071118	5540051018	5040030622	5040030620	5035020422	601222	551220	501020	501020	501020	451020	601420	551220	501020	501020	501020	451020	38170
8	5545051018	5545051018	5540050918	5040040818	5040030620	4540030620	601420	551420	551220	501020	501020	451020	601420	551420	551220	501020	501020	451020	38531
9	5545051018	5545051018	5540050918	5040050818	5040030620	4540030620	601420	551420	551220	501020	501020	451020	601420	551420	551220	501020	501020	451020	38588
10	5545051018	5545051018	5540050918	5040050918	5040030620	4540030620	601420	551420	551220	501020	501020	451020	601420	551420	551220	501020	501020	451020	38595

Figure 8. A part of the created database for the 6-story structure with the values of ρ of columns 2%≤ρ< 2.5%

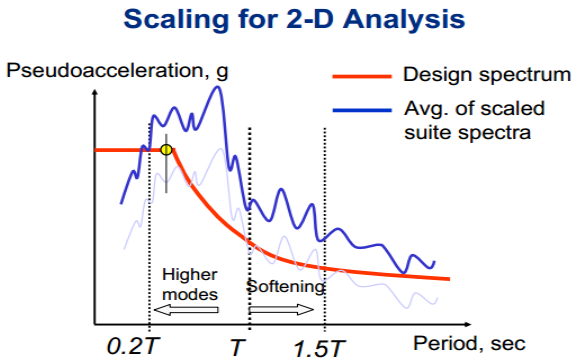


Figure 9. Schematic chart of scaling the accelerograms [22]

States [23-25], and it was computed by the program for each section of the columns, and then used in the non-linear time-history analyses [26].

5. 4. Selecting the Parameters for Sensitivity Analysis The parameters selected for sensitivity analysis consist of the story maximum plastic drift, and the maximum rotation of the plastic hinges in the beams and columns.

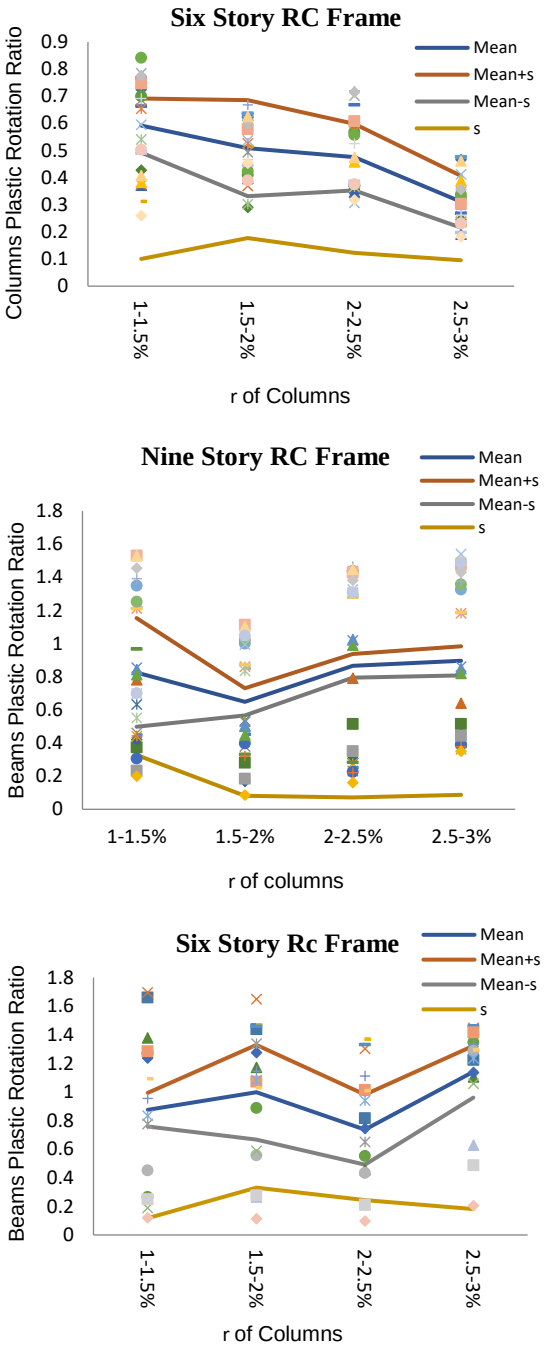
5. 5. Analysis of the Main Parameters Affecting the Structure Robustness Non-linear time-history analysis was conducted for the created databases of the structures, and the median of maximum values of parameters mentioned in the last section was extracted for each of the structural members based on the selected records. In order to obtain the dimensionless form of the data, these results were divided by their acceptance criteria values in the non-linear analyses according to ASCE/SEI 41-13 [20].

5. 6. Calculation of the Standard Deviation The standard deviation and mean was calculated for the median of maximum values of the selected parameters in each one of the statistical populations, and the chart for the average variations of the answers was drawn based on the input parameters variations together with the standard deviation of the outputs. The standard deviation equation is:

$$\sigma = \sqrt{\frac{1}{N}\sum_{i=1}^{N}(x_i - \mu)^2} \quad (18)$$

In which σ is the standard deviation, μ, mean of the data, N, number of the statistical population, and x is the data.

5. 7. Results from the Statistical Analyses of the Non-linear Time-history Analysis Output According to the discussions above, the output of non-linear time-history analysis is consisted of the maximum plastic inter-story drift, and maximum rotation of the plastic hinges in beams and columns of all stories. Here, at the first, the output of local data, i.e. maximum rotation ratio of plastic hinges in the beams and columns are presented according to the variations of the reinforcement ratio of columns. Next, the outputs related to the maximum rotation ratio of plastic hinges in the beams and columns according to the variations of the reinforcement ratio of beams are presented in Figure 10.



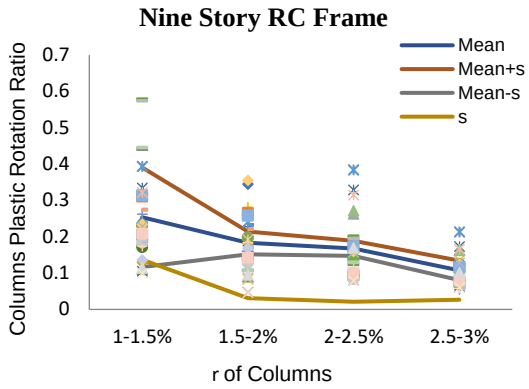
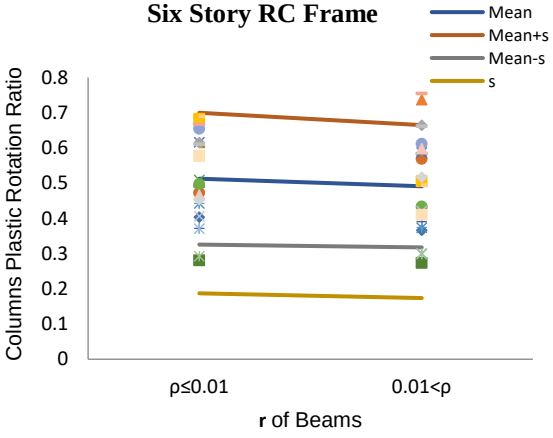


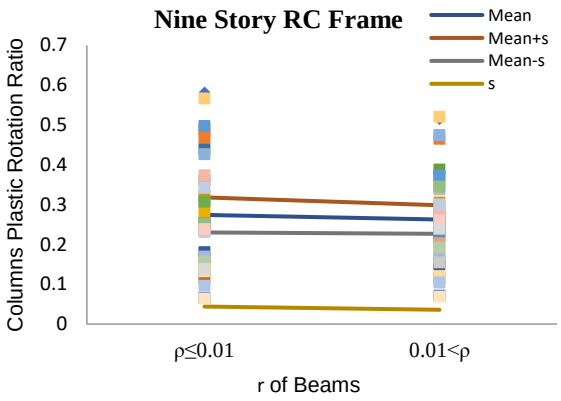
Figure 10. Charts of the average variations of the plastic hinges rotation ratio in beams and columns, and their variance according to the variations of the reinforcement ratio of columns in the 6-story & 9-story building

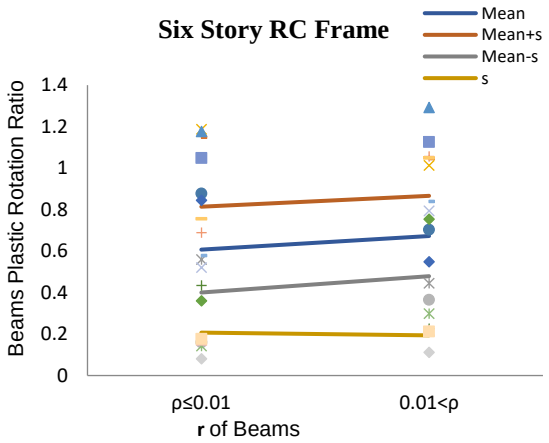
As understood from Figure 10, with an increase in the reinforcement ratio of the columns, the plastic hinges rotation in the columns decreases, and the plastic hinges rotation in the beams increases. This means that the failure mechanism moves towards the formation of the beam mechanism. In addition, the variance of plastic rotations also decreases with the increase in the reinforcement ratio of the columns. This results in an increase in the predictability of structural behaviour.

It can be seen in Figure 11 that with an increase in the reinforcement ratio of the beams, the plastic hinges rotation in the columns decreases, and the plastic hinges rotation in the beams increases. This shows that plastic hinges are first formed in beams. In addition, the variance of the plastic hinges rotations also decreases with an increase in the reinforcement ratio of the beams, and as a result, the predictability of structural behaviour increases.

As a global criterion in Figure 12, it can be seen that by increasing the ratio of steel in beams and columns, the behaviour of the structure improves.







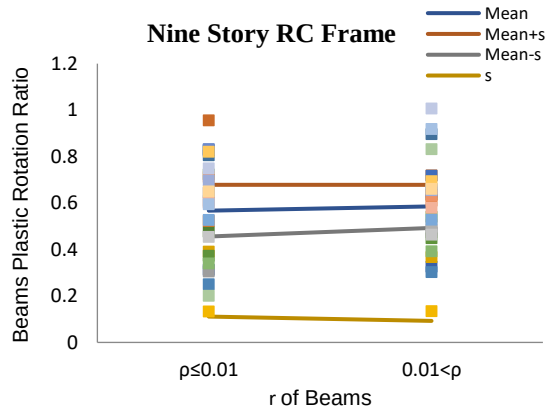


Figure 11. Charts of the average variations of the plastic hinges rotation ratio in beams and columns, and their variance according to the variations of the reinforcement ratio of beams in the 6-story & 9-story building

Therefore, with an increase in the ratio of reinforcing bars in the beams and columns, the probability of the formation and development of plastic hinges in the beams increases, while it decreases in the columns. As shown schematically in Figure 13.

The inter-stories drift of the structures resulted from non-linear time-history analysis, is presented above as a global criterion.

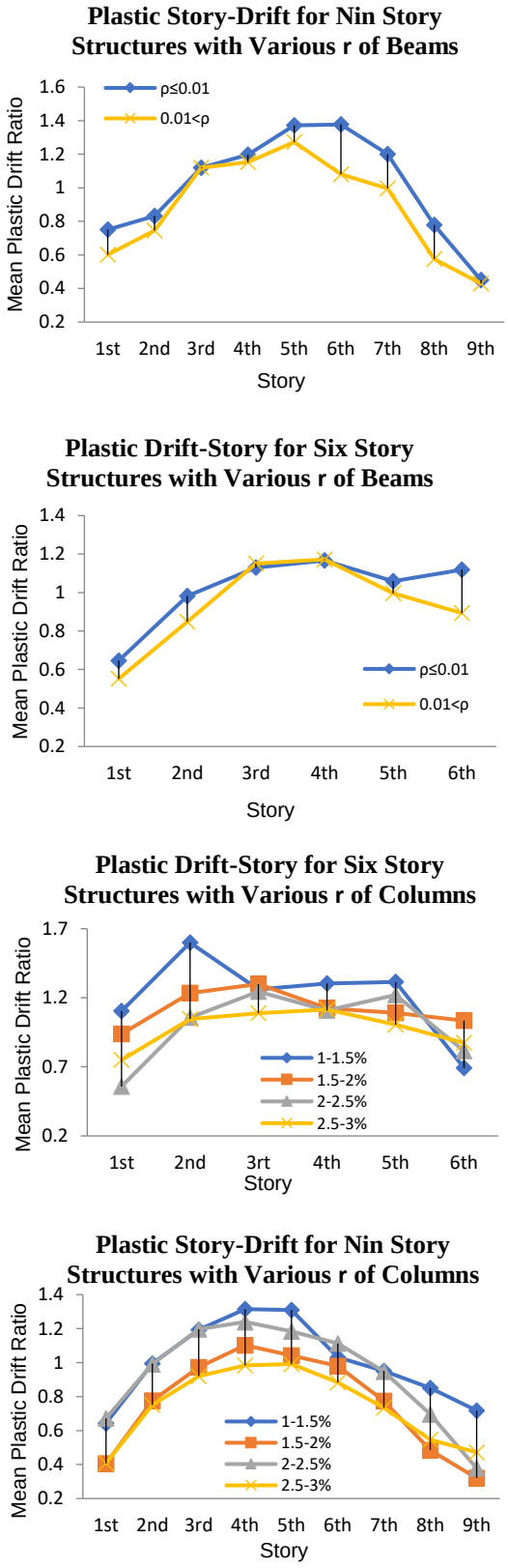


Figure 12. Chart of plastic drift ratio of the stories for the 6 & 9-story structures with various values of reinforcement ratio of beams & columns

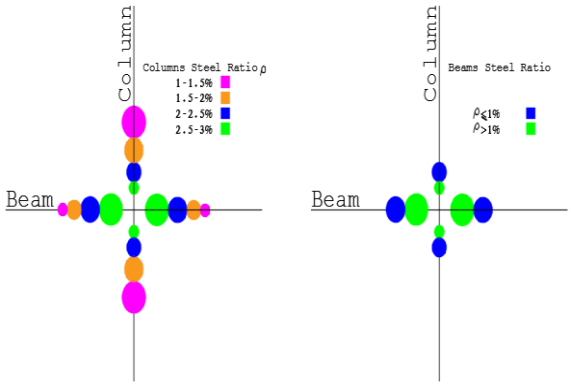


Figure 13. Variation of the probability of plastic hinges formation and development in the beams and columns, together with the variation of the reinforcement ratio in the beams and columns

6. CONCLUSION

The probability of failure can be significantly reduced for the structures which have adequate robustness. In fact, performance assessment of the designed structures by the practicing engineers under probable earthquakes has complicated steps to limit the possible damages.

In this study, an attempt is done to propose a methodology to design robust RC frames, which results in improving seismic performance of these structures. To do so, certain ranges are selected for the design parameters and several structures are designed using optimization algorithms within these ranges, and structural response evaluated by nonlinear time-history analysis are statistically analysed to assess the robustness of the optimal designs.

An increase in the probability of hinge formation in the beams leads to the increment of the probability of formation of beam mechanism, and the decrease in the variance of results also shows the increasing in the predictability of structural behaviour, both of which would lead to increased robustness of the structure.

According to the results of local and global criteria, it can be concluded that structural engineers, without engaging in complex calculations, can obtain structures with higher robustness by using sections sticking to the high steel ratio limits.

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Persian Abstract

چکیده

قابهای خمشی بتن‌آرمه (RCMRF) از سیستمهای سازه‌ای هستند که کاربرد گسترده‌ای دارند. اصولاً ساختمان‌های با سیستم RCMRF به نحوی طراحی می‌شوند که نیازهای تغییر مکان نسبی، مقاومت و انعطاف پذیری، تعیین شده در آئین نامه‌های طراحی، را برآورده نمایند. در روند طراحی این سیستمها محدوده‌هایی برای پارامترهای طراحی در آئین نامه‌ها ارایه شده‌اند که مهندسین براساس آنها سازه‌ها را طراحی می‌کنند و مقادیر انتخابی برای این پارامترها بر روی رفتار لرزه ای سازه ها مؤثر است. ولی بررسی عملکرد سازه‌های طراحی شده توسط مهندسین، تحت زلزله‌های محتمل به منظور محدود نمودن خسارات احتمالی و کاهش ضعفهای موضعی و افزایش تحمل سازه در برابر زلزله دارای مراحلی بس دشوار و پیچیده است که طی این مراحل برای طراحی ساختمانهای معمولی زمان‌بر است. در این تحقیق محدوده‌هایی تنگ‌تر برای پارامترهای طراحی تعیین شده که در صورت رعایت این محدوده‌ها در زمان طراحی، مهندسین به طرحهایی دست خواهند یافت که عملکرد لرزه ای آنها بهتر باشد. در این راستا برای دامنه های مختلف پارامترهای طراحی، پایگاههای داده ای برای RCMRF ها ایجاد شد. در ایجاد این پایگاه داده ها از الگوریتم بهینه‌سازی دسته ذرات (PSO) استفاده شد و RCMRF ها بر اساس آئین نامه ACI318-14 بصورت بهینه طراحی شدند سپس سازه‌های بهینه طراحی شده براساس آئین نامه ASCE/SEI 7-16 تحت تحلیل غیر خطی تاریخچه–زمانی قرار گرفتند و تحلیلهای آماری برورودی نتایج محلی و کلی جامع حاصله از تحلیل غیر خطی انجام یافت. و نهایتاً براساس نتایج این تحلیلهای آماری، محدوده های تنگتری به منظور دستیابی به سازه های با استواری بالاتر بدون درگیر شدن در فرآیندهای زمان بر تعیین گردید.