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Optimum seismic design of reinforced concrete frame structures

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Abstract. This paper proposes an automated procedure for optimum seismic design of reinforced concrete (RC) frame structures. This procedure combines a smart pre-processing using a Tree Classification Method (TCM) and a nonlinear optimization technique. First, the TCM automatically creates sections database and assigns sections to structural members. Subsequently, a real valued model of Particle Swarm Optimization (PSO) algorithm is employed in solving the optimization problem. Numerical examples on design optimization of three low- to high-rise RC frame structures under earthquake loads are presented with and without considering strong column-weak beam (SCWB) constraint. Results demonstrate the effectiveness of the TCM in seismic design optimization of the structures.

Keywords: automated optimum seismic design; reinforced concrete structure; tree classification method; construction cost; strong column-weak beam; particle swarm optimization

1. Introduction

The objective of the structural engineer is to design structures with an acceptable performance level against possible future earthquakes accompanied with a minimum construction cost (Kanno and Takewaki 2007, Plevris *et al*. 2012, Moustafa 2013). Conventional design based on trial and error procedure guided by instinct and experience may not be a suitable tool, particularly for large scale and complex structures. A notable example is multi-storey multi-bay structures having high redundancy. Herein, obtaining a safe and an economic design may not be achieved without employing the structural optimization. Given recent advances in computers and computational techniques, this rather complicated and challenging problem should be replaced with a computer-automated design procedure within the framework of structural optimization (Fragiadakis and Lagaros 2011, Lagaros 2014).

Structural design optimization deals with minimizing or maximizing a single or multiple

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objective functions (such as the structure's weight, construction cost, seismic input energy, hysteretic energy dissipation, etc.) while maintaining a set of predefined constraints. Applications of structural optimization includes the optimal design of truss structures, frame buildings, shells, plates, dams, bridges, nuclear structures, storage tanks and retaining walls, etc. The optimal design of frame buildings (steel or RC structures) represents one of the ongoing research topics for the past few decades or so. In steel structures, the number of design variables is relatively small. In contrast, the optimal design of RC structures involves a large number of design variables and constraints (Gharehbaghi and Fadaee 2012). Accordingly, a small number of benchmark examples on optimization of RC structures is available compared particularly with steel truss and frame structures.

Recent developments in design optimization of RC structures include employing different techniques to solve the problem of optimal design of structures. The Linear and Non-Linear Programming, and the Optimality Criteria techniques were used to find the optimal design of RC frame structures under static loads (Krishnamoorthy and Munro 1973, Gerlein and Beaufait 1980, Moharrami and Grierson 1993, Fadaee and Grierson 1996, Balling and Yao 1997, Fadaee and Grierson 1998, Guerra and Kiousis 2006). The Genetic Algorithms were also used to solve the same problem (Rajeev and Krishnamoorthy 1989, Lee and Ahn 2003m Camp *et al.* 2003, Kwak and Kim 2009). The Big Bang-Big Crunch algorithm was also applied to minimize the construction cost of RC frames (Kaveh and Sabzi 2011, Kaveh and Sabzi 2012, Camp and Huq 2013). The Bat algorithm has been utilized in the design optimization of RC frames (Gholizadeh and Aligholizadeh 2013). Similarly, the Charged System Search (CSS) has been used in design optimization of RC frames (e.g. Kaveh and Behnam 2013). Kao and Yeh (2014) have also proposed and assessed a five-phase method which integrates Design, Analysis, Modeling, Definition, and Optimization (DAMDO) phases into an integration environment. The real valued model of Particle Swarm Optimization (PSO) algorithm has been utilized to design optimization of RC framed structures under time-history earthquake loads (Gharehbaghi *et al.* 2011; Gharehbaghi *et al.* 2012, Gharehbaghi and Fadaee 2012, Gharehbaghi and Khatibinia 2015). The optimum seismic design of RC structures has attracted much attention worldwide (Zou and Chan 2005, Lagaros and Papadrakakis 2008, Gharehbaghi *et al.* 2011a, Gharehbaghi *et al.* 2011b, Gharehbaghi and Fadaee 2012; Kaveh and Zakian 2014a, Kaveh and Zakian 2014b, Gharehbaghi and Khatibinia 2015, Khatibinia *et al.* 2015, Moustafa 2015).

The objective of this study is to minimize the construction cost of low- to high-rise RC frame structures by using a new simple and efficient computer-automated methodology using structural optimization. The objective is to obtain the best combination of members' cross-section dimensions and steel reinforcement within the possible feasible region. In this paper, the successful real valued version of PSO algorithm (Gharehbaghi *et al.* 2011, Gharehbaghi *et al.* 2011, Gharehbaghi and Fadaee 2012, Gharehbaghi and Khatibinia 2015) is used to implement the optimization problem. Herein, the initial construction cost is considered as an objective function. The constraints of the optimization problem conform to the provisions of the American Concrete Institute (ACI318) (2011) and International Building Code (IBC) (2012). In general, the design constraints can be classified into three groups: (i) practical restrictions related to allowable section and elements condition and structural configuration, (ii) capacity criteria and seismic provisions for combination of static loads, and (iii) capacity criteria and seismic provisions for combination of gravity and earthquake loads. In most of engineering optimization problems, reducing the number of design variables and constraints can reduce the computational efforts of the optimization problem. Hence, this paper focuses on elimination of the number of constraints and virtually reducing the number of design variables by proposing a new methodology termed as Tree Classification Method (TCM). TCM deals with the construction of a comprehensive section database and assigning sections to structural members. This method is carried out before performing the optimization procedure by an automated and user-defined pre-processing technique. This, in turn, leads to a significant reduction in the number of design constraints. Also, the real design variables are converted to pseudo-design variables, and, thus, the number of design variables is significantly reduced. Accordingly, the computation and search time to reach the optimum design can be significantly reduced. To verify the effectiveness of the proposed methodology, two RC frame structures including six and twelve-storey buildings with three bays are optimally designed considering the seismic equivalent static analysis method.

An important design concept recommended in ACI-318 (2011) is the strong-column weak-beam (SCWB) concept. This constraint has a significant effect on the design and performance of frame structures to earthquake loads. Herein, the problem of optimal design of RC frame structures is solved with and without considering the SCWB constraint. It may be noted that the optimal design of large scale structures is a computationally intensive task particularly under time-history earthquake loads (Papadrakakis *et al*. 2001a, Papadrakakis *et al*. 2002). Thus, reducing the number of analyses to achieve the optimum design is of crucial importance. In this paper, the design optimization of a high-rise RC frame structure having three bays and eighteen stories subjected to three code-compatible artificial ground motion records is presented to illustrate the efficacy of the proposed methodology. This frame is optimally designed for seismic equivalent static loads and the results are compared to that of the frame optimized for time-history earthquake loads. The next section presents the formulation of design optimization of RC structures under seismic loads.

2. Optimization problem

2.1 Numerical simulation procedure

Due to the material capacities, deformation restrictions and cost limitations, the optimal design of RC frame structures is a constrained optimization problem. Mathematically, a constrained optimization problem can be expressed as (Papadrakakis *et al*. 2001b)

$$\begin{gathered} \text{Minimize } F(x) \\ \text{subjected to } g_i(x) \leq 0.0 \;\; i = 1, 2, ..., m; \\ x_j \in R^d,\; j = 1, 2, ..., n \end{gathered} \tag{1}$$

where $F(x)$ is the objective function, $g_i(x)$ is the ith constraint; m and n are the number of constraints and the design variables, respectively; R^d is a given set of discrete real quantities from which the design variables x_j take values. To convert the constrained structural optimization problem into unconstrained problem, a penalty function method is used by constructing a new weighted function as follows (Goldberg 1989, Khatibinia *et al*. 2015)

$$\Phi(x, r_p) = F(x) + r_p \times \sum_{i=1}^{m} (\max\{g_i, 0.0\})^2 \tag{2}$$

where Φ and r_p are the pseudo objective function, and positive penalty ratio, respectively. In structural design optimization, the objective is typically to minimize the structural weight or the construction cost of the structure under a set of pre-defined constraints. This formulation generates solutions with violated constraints, and the objective function is greater than the non-violated value for the generated design variables.

In design optimization of structures made of two or more materials, minimum weight has no meaning with respect to optimization. Hence, in this paper, the objective function is taken as the initial construction cost of the structure, defined as

$$Construction\,Cost = \sum_{i=1}^{ne}(C_{ci}A_{ci}L_i + C_{si}A_{si}L_i + C_{fi}A_{fi}) \tag{3}$$

where C_{ci} and A_{ci} are the cost per unit volume and total area of the cross-section of *i*th element related to concrete, respectively. C_{si} and A_{si} are the cost per unit weight and area of steel bars in the cross-section of *i*th element, respectively; C_{fi} and A_{fi} are also the cost per unit area of formwork and its area in the cross-section of *i*th element, respectively; L_i is the length of *i*th element; and *ne* is the number of structural elements. In RC structures, however, three cost components due to concrete, steel reinforcement and formwork should be considered. Herein, the cost units are assumed to be C_c=735\$/m^3, C_s=55735\$/m^3 and C_f=54\$/m^2 (Camp and Huq 2013).

2.2 Optimization problem constraints

In sizing optimization problems of structures, design criteria are applied as the problem constraints. In this paper, the design criteria are schematized in three sets of constraints. The first set of constraints is related to the practical aspects and preliminary cross-section conditions. The second set is the constraints employed to control the capacity of beams and columns against the combination of gravity and static lateral loads, consideration of allowable SCWB ratio and storey drift. The third set includes the constraints utilized to check the capacity criteria and seismic provisions due to gravity and earthquake loads.

2.2.1 The first set of constraints

To design the structural elements, these constraints depend on the practical aspects, and preliminary cross-section conditions are considered based on ACI318 (2011) design code. These constraints are expressed as follows

$$\rho_{\min} = \max(\frac{1.4}{f_y}, \frac{0.25\sqrt{f'_c}}{f_y}) \le \rho_b \le \rho_{\max} = 0.85\beta_1\frac{f'_c}{f_y}\frac{\varepsilon_{cu}}{\varepsilon_{cu}+0.004}) \tag{4}$$

$$1\% \le \rho_c \le 4\% \tag{5}$$

$$ds = \frac{b - 2c - 2d_{bt} - n'_{bl}d_{bl}}{n'_{bl} - 1} \ge ds_{all} \tag{6}$$

$$\{b^t, h^t, n^t_{bl}, d^t_{bl}\} \le \{b^b, h^b, n^b_{bl}, d^b_{bl}\} \tag{7}$$

where ρ_c, ρ_b, ρ_{min} and ρ_{max} represent the reinforcement percent of cross-section of columns, the reinforcement percent of cross-section of beams, the minimum and maximum allowable reinforcement percent of cross-section of beams, respectively; c, b, d_{bt}, and d_{bl} are cover and width of cross-section, diameter of transverse reinforcement, and diameter of longitudinal reinforcements, respectively. Also, n'_{bl} is the number of longitudinal reinforcements in each side of the cross-section. $f_c^{'}$ is concrete strength in compression, f_y is the yield stress of steel bars, β_1 is a positive coefficient, and ε_{cu} is the ultimate compressive strain of concrete. Also, in Eq. (7), b, h, d_{bl} and n_{bl} (with superscripts t and b represent the top and bottom of a storey level) are the width, depth, diameter and number of longitudinal reinforcements for both beams and columns which are in same direction between two subsequent storeys, respectively. Also, ds and ds_{all} are the spacing between parallel longitudinal bars in a layer and its allowable value.

The value of ds_{all} for columns and beams appearing in Eq. (6) above is defined as

$$ds_{all} = \begin{cases} \max\{25mm, d_{bl}, 1.33d_{\max}\} \text{ for Beams} \\ \max\{40mm, 1.5d_{bl}, 1.33d_{\max}\} \text{ for Columns} \end{cases} \tag{8}$$

where d_{max} is the diameter of greatest aggregate of concrete.

ACI318 (2011) also recommends considering the restrictions on the minimum and maximum values of the width and depth of cross-section of beams (i.e., the aspect ratio of the cross-section dimensions to avoid deep beam) encapsulated as

$$h_{b\min} = 0.0476L_b < h_b < h_{b\max} = 2.5b_b \tag{9}$$

in which h_b, b_b and L_b are the depth, width and length of beams.

2.2.2 The second set of constraints

The second set of constraints is considered for controlling the capacity of beams and columns against the combination of gravity and seismic equivalent static lateral loads. This can be represented as follows

$$M_u^{\ b} \le \phi_b M_n^{\ b} \tag{10}$$

in which $M_u^{\ b}$, $M_n^{\ b}$ and ϕ_b are the factored externally applied moment, nominal flexural strength and strength reduction factor for beams, respectively. The value of ϕ_b is considered equal to 0.9.

To examine the capacity of columns under gravity loads, the combination of axial load and bending moment applied to the cross-section of columns should be applied. An idealized P-M interaction curve with characterized points was introduced in the literature. More details on the characterized points can be found in (Gharehbaghi and Fadaee 2012). Herein, the idealized P-M curve is utilized for controlling columns capacity. Mathematically, using P-M curve, the following expression is reached:

$$\sqrt{(M_u^{\ c})^2 + (P_u^{\ c})^2} \le \sqrt{(\phi_c M_n^{\ c})^2 + (\phi_c P_n^{\ c})^2} \tag{11}$$

where $M_u^{\ c}$, $M_n^{\ c}$, $P_u^{\ c}$, $P_n^{\ c}$ and ϕ_c are the factored externally applied moment, nominal flexural strength, factored externally applied axial force, nominal axial strength and strength reduction factor for columns, respectively. The value of ϕ_c is varied from 0.65 to 0.90 based on ACI318 (2011).

As seismic provisions, two other constraints shall be considered. First, based upon ACI318 (2011), the SCWB concept should be satisfied particularly in seismically active regions using the following inequality

$$\frac{\sum M_{c\,j}}{\sum M_{b\,j}} \geq 1.2 \tag{12}$$

in which $\sum M_{c\,j}$ is the sum of moment capacity of columns at the top and bottom faces of jth structural joint; $\sum M_{b\,j}$ also, is the sum of the moment capacity of beams at the left and right of jth structural joint. The inequality shall be satisfied for all of the structural joints. Second, one of the most important design constraints subjected to lateral loads is the inter-storey drift ratio (ISDR). According to the recommendations of IBC (2012), another restriction shall be satisfied in terms of the storey drift as follows

$$\frac{\Delta_i}{h_i} \leq 0.025 \tag{13}$$

where Δ_i and h_i represent storey drift and height of ith storey of structure.

2.2.3 The third set of constraints

When the structure under consideration is subjected to strong ground motions represented by the time-history of the ground accelerations, the constraints expressed in the form of Eqs. (10) and (11), should be checked for dynamic effects due to the loads. Hence, the capacity of beams and columns should be checked for critical conditions. In the case of beams, the critical condition is defined as the maximum of externally applied moment during time-history loads. Also, in the case of columns, the critical condition is devoted to the critical combination of axial load and bending moment applied to the cross-section that can be defined as a function depending on time. Accordingly, Eqs. (10) and (11) can be generalized for beams and columns respectively, as follows (Gharehbaghi and Fadaee 2012)

$$\max(M_u{}^b(t)) \leq \phi_b M_n{}^b \tag{14}$$

$$\max\left(\sqrt{(M_u{}^c(t))^2 + (P_u{}^c(t))^2}\right) \leq \sqrt{(\phi_c M_n{}^c)^2 + (\phi_c P_n{}^c)^2} \tag{15}$$

in which, t is the time of ground motion record. It is evident that the maximum combination of axial load and bending moment at columns called critical conditions should not be considered by the combination of the maximum of bending moment and the maximum of axial loads during earthquake simultaneously (Gharehbaghi and Fadaee 2012). Also regarding the ISDR criteria, the following constraint shall be assessed when the time-history earthquake loads are considered (IBC 2012)

$$\frac{\max(\Delta_i(t))}{h_i} \leq 0.025 \tag{16}$$

In fact, this equation is to check the maximum time-depended storey drift ratio respected to its allowable value.

2.3 PSO algorithm

A well-known optimization algorithm, namely PSO, was presented to simulate the motion of bird swarms as a part of a socio-cognitive research in the mid 1990s (Kennedy and Eberhart 2001). PSO has been motivated by the communal actions of such animals as fish schooling, insects swarming and birds flocking. It involves a number of particles initialized randomly in the feasible search space of the problem domain. These particles are referred to as swarm. Each particle of the swarm represents a potential solution of the optimization problem. The particles fly through the search space and their positions are updated based on the best positions of individual particles in iterations. The objective function is assessed for each particle and the fitness values of particles are achieved to find out which position in the search space is the most excellent (Bergh and Engelbrecht 2003). In iteration k, the velocity and position vectors of a swarm are updated using the following equations

$$\boldsymbol{V}_i^{k+1} = w^k \boldsymbol{V}_i^k + c_1 r_1 (\boldsymbol{P}_i^k - \boldsymbol{X}_i^k) + c_2 r_2 (\boldsymbol{P}_g^k - \boldsymbol{X}_i^k) \tag{17}$$

$$\boldsymbol{X}_i^{k+1} = \boldsymbol{X}_i^k + \boldsymbol{V}_i^{k+1} \tag{18}$$

where $\boldsymbol{Xi}^k$ and $\boldsymbol{Vi}^k$ represent the current position and the velocity of the ith particle, respectively; $\boldsymbol{P}_i^k$is the best previous position of the ith particle (called p_{best}) and $\boldsymbol{P}_g^k$ is the best global position among all the particles in the swarm (called g_{best}); $r_1$and r_2 are two uniform random sequences generated from interval [0, 1]. Shi and Eberhart (1997) also proposed that the cognitive and social scaling parameters c_1 and c_2 can be selected such that $c_1= c_2=2.0$ to allow the product c_1r_1 or c_2r_2 to have a mean of 1.0; w^k is the inertia weight used to discount the previous velocity of particle preserved. As an optimal updating criterion, w^k has a key role in updating velocity and position vectors leading to an efficient search behavior and is taken as follows (Gholizadeh and Salajegheh 2009)

$$w^k = w_{\max} - \frac{w_{\max} - w_{\min}}{k_{\max}} k \tag{19}$$

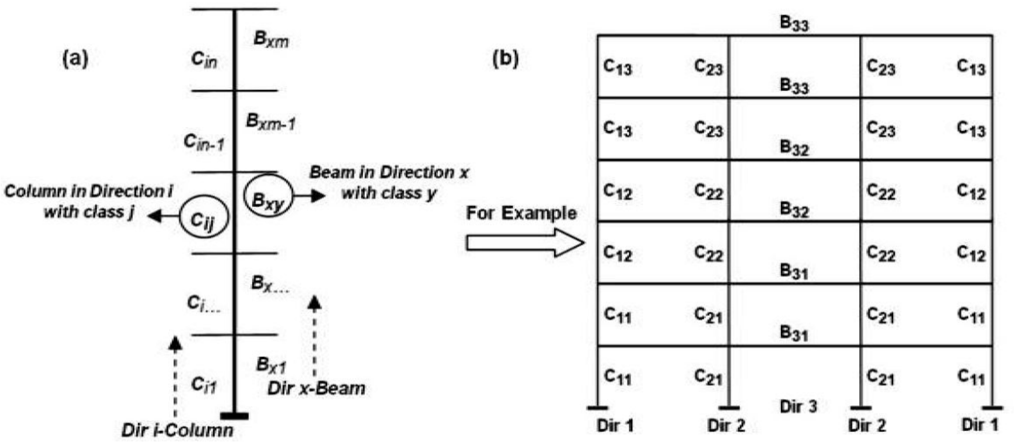
in which w_{max} and w_{min} are the upper and lower bounds of w, respectively. Also, k_{max} is the number of maximum considered iterations. Based on a sensitivity analysis conducted for optimal design of RC frames, in this paper, the best values of 0.004 and 0.009 are considered for w_{min} and w_{max} respectively.

The successful applications of the binary PSO algorithm (Kennedy and Eberhart 2001) in structural optimization under static and earthquake loads were reported in literature (e.g., Salajegheh *et al*. 2008, Seyedpoor *et al*. 2011, Gholizadeh 2013). In the current study an improved version of the PSO algorithm (i.e. the real valued model of PSO) is employed to implement the optimization procedure. The successful applications of this version of PSO can be found in (Salajegheh *et al*. 2008, Gharehbaghi *et al*. 2011a, Gharehbaghi *et al*. 2011b, Gharehbaghi and Fadaee 2012, Gharehbaghi and Khatibinia 2015). In this model, the decimal values of the design variables are utilized in the optimization process instead of their binary codes. Accordingly, the length of the particles is shortened, and, therefore, the convergence of the algorithm can be achieved with lower computational effort and higher speed.

It should be noted that, although there are new meta-heuristic optimization algorithms having relatively better performance with respect to PSO, the main contribution of the paper is to show the effects of using TCM, as a pre-processing procedure, on the optimal design process of RC frame structures. In the other word, authors have not focused on the utilizing the recently proposed algorithms herein. The next section describes the TCM and solution procedures.

3. The proposed optimization methodology

In solving the structural optimization problem, one should precisely define the objective function, the optimization variables, and the associated constraint. Decreasing the number of optimization variables and/or the constraints could significantly increase the convergence rate to the global optimum and thus the optimal solution could be reached faster. In general, many constraints have a role in the optimal seismic design of RC structures due to controlling the practical limitations, capacity criteria, and seismic provisions. For instance, the practical constraints summarized in Eq. (7) have a remarkable effect on the convergence rate of the optimal solution. In this paper, a simple but useful technique is proposed to reduce the design constraints whereas the role of cross-section conditions and practical limitations are eliminated within the optimization process by a user-defined pre-processing. These constraints encapsulated as the first set of constraints can be divided into two cases: case 1, the restrictions related to the details of cross-section of beams and columns based on ACI318 (2011) summarized in Eqs. (4) to (6), (8) and (9); and case 2, some limitations based on engineering judgments summarized in Eq. (7). To achieve this goal, a code-programming is developed by separately defining two section databases for beams and columns. These databases include a range of the width and depth of cross-section dimensions, a range of different steel bars, and a range of different number of steel bars at the top and bottom of cross-section. Depending on the configuration of the considered structure in terms of classification of beams and columns over the height, the number of directions for beams and columns can be considered. As shown in Fig. 1a, a structural tree is consisted of columns with direction "i" and beams with direction "x".



(a) A structural tree (b) Multi-storey frame example for TCM

Fig. 1 Tree Classification Method for optimal design of RC frame structures

Within the direction "*i*", different classes of columns from *1* to *n* exist with identification label of C_{ij} that is the columns in direction *i* with class *j*. Similarly, as depicted in Fig. 1(a), within the direction "*x*", different classes of beams from *1* to *m* also exist with an identification label of B_{xy} that is the beams in direction *x* with class *y*. Note that, directions of beams are separated from directions of columns. However, in optimal design of frame structures the same direction of beams and columns could exist. This concept is illustrated for a three-bay six-storey frame structure in Fig. 1b. Herein, two different directions (C_{1j} and C_{2j}) are assigned to columns and one direction is assigned to beams (B_{3y}).

Earlier researches (e.g. Kaveh and Sabzi 2012; Camp and Huq 2013; Gholizadeh and Aligholizadeh 2013; Kaveh and Behnam 2013; Gharehbaghi and Khatibinia 2015) have used similar planning (known as conventional method herein) for element's classification. Here, the focus is on a tree of elements in each direction as shown in Fig. 1. In fact, corresponding to the number of classes of elements over each direction of the structure, each row of section database is defined while all of the limitations summarized in Eqs. (4) to (9) are controlled to be allowable. For example, each row of column database is consisted of *n* number of sections by satisfying Eqs. (4) to (9) while as mentioned above, *n* is the number of classes of columns within the considered direction. Hence, the pre-processing step is carried out before starting the optimization process, and thus the section databases are constructed offline. Subsequently, according to the number of independent directions of the structure, the randomly selected rows of database are assigned to each direction of structure in order to create a preliminary design candidate. This process is executed on initial particles (structural candidates) defined for starting the PSO algorithm. In effect, these initial guesses of the structural candidates and their updated population in the next iterations of optimization process are acceptable in terms of first set of design constraints. The entire pre-processing is adopted using a code-programming written in MATLAB platform (2010). The flowchart depicted in Fig. 2 demonstrates the steps of the proposed computer-automated optimization process.

Based on the optimization concepts, as shown in Fig. 1b, there are three pseudo-design variables corresponding to the three different existing directions in this frame. On the other side, different classes of elements are located over each direction. For instance, the mentioned frame has nine classes of elements (six classes for columns and three classes for beams); as a result, it has nine real design variables. Fig. 3(a) shows a scheme for TCM describing how it converts the real to pseudo-design variables. Also, Fig. 3(b) and Fig. 3(c) compare the way of data sampling from section database using the conventional methods in the literature with proposed TCM. So, according to the proposed idea as shown in Figs. 1 and 3, three number of pseudo-design variables is used instead of nine number of real design variables, and it may be said that the number of design variables is virtually decreased from nine (classes) to three (directions). In fact, based on the TCM role depicted in Figs. 1 and 3, the number of initial sampling data which are randomly selected from section database is equal to the number of direction or on the other word pseudo-design variables. Also, during the optimization process using PSO algorithm, the position ($\boldsymbol{X_i}$) and velocity ($\boldsymbol{V_i}$) vectors of *i*th particle has a length equal to the number of directions or pseudo-design variables. So, the use of TCM technique reduces the number of design constraints. Also, as characterized in Fig 3a, the technique virtually reduces the number of design variables that is shortening the length of the position ($\boldsymbol{X_i}$) and velocity ($\boldsymbol{V_i}$) vectors. As a result, the idea could increase the convergence rate of the optimization process. In the following Sections, the effectiveness of the proposed methodology is discussed. The structures studied and applied earthquake loads are explained in the next section.

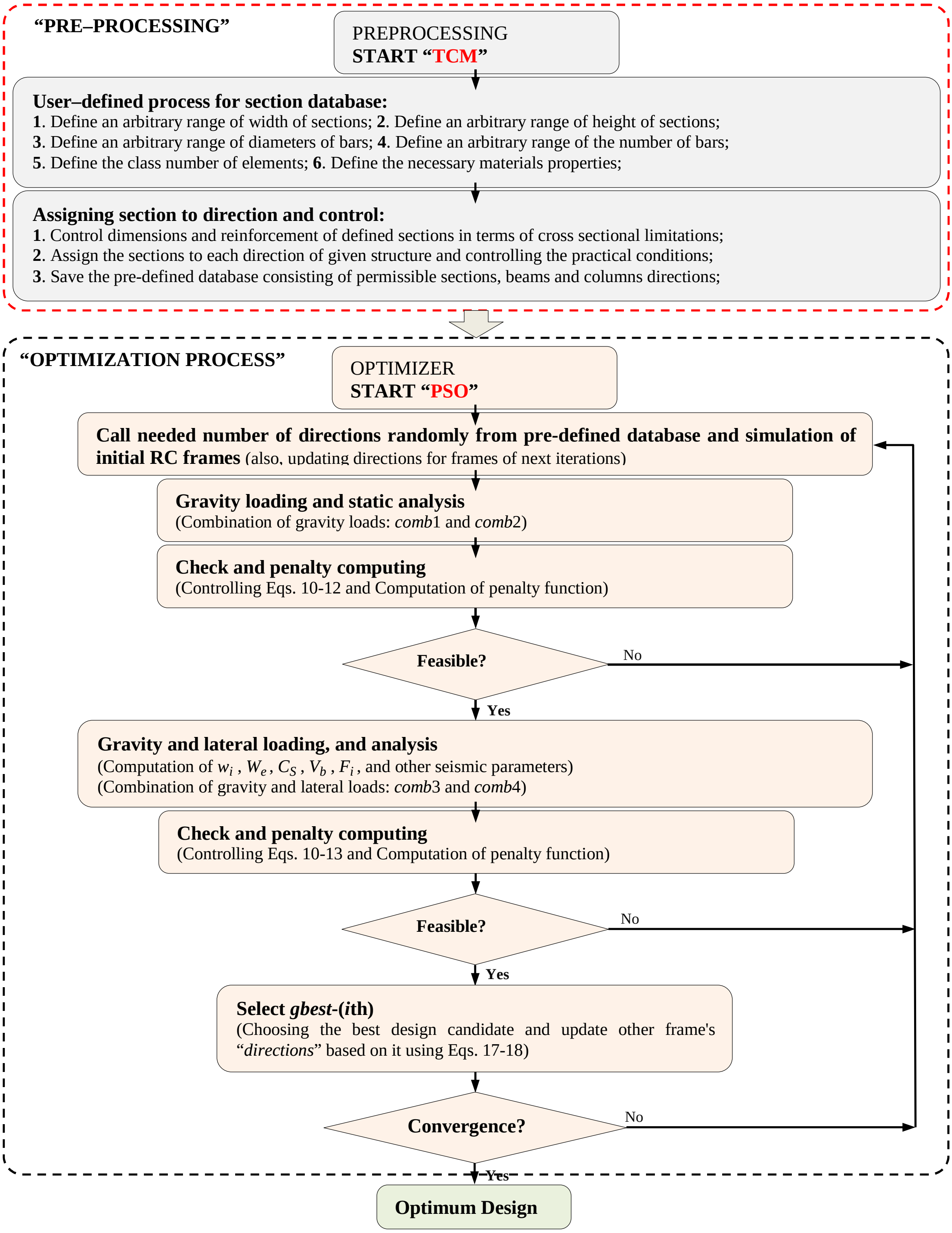
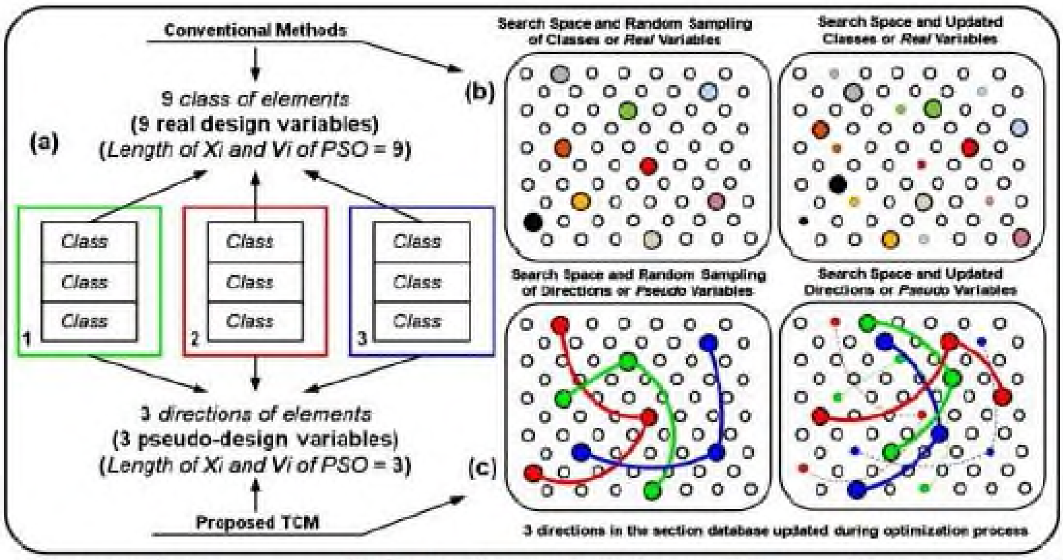


Fig. 2 Flowchart of the proposed TCM for optimal seismic design of RC structures



(a) converting to pseudo-design (b) Conventional methods for sampling classes and updating them (c) Proposed TCM for sampling directions

Fig. 3 A scheme for TCM role

4. The structures considered, assumptions and loadings

4.1 Frame's configuration and pre-defined section databases

Six-, twelve- and eighteen-storey of three bays RC frame structures (abbreviated here as 6S-3B, 12S-3B and 18S-3B, respectively) are considered to illustrate the proposed optimization methodology and to examine its efficacy. These frames represent low, medium and high-rise buildings. As shown in Fig. 4, the beams and columns are classified separately using the proposed TCM method.

Previous to starting the optimization process, pre-defined section database are generated. Two databases for beams and columns are independently produced. These databases include several rectangular sections conforming to ACI318 (2011) design code recommendations. For beam sections, the depth of sections is taken to range from 400 to 900 mm associated with three widths of 350, 400 and 450 mm. Furthermore, in the numerical simulation of the cross-sections databases, the difference between the dimensions of sequential sections is taken as 50 mm. For columns, the same numerical values of depth and width ranging from 400 to 900 mm are used.

The topology and arrangements of steel bars is illustrated in Fig. 5 for columns (two cases) and beams (three cases). Note that, regarding the center region of the beams the reverse arrangement of bars is assumed based on the tension region caused by the bending moment. The diameter of longitudinal reinforcements is taken between 12 and 28 mm in the databases with a step of 2 mm between each two consecutive bars. Transverse bars (stirrups) of 10 mm diameter are taken to resist shear stresses. The minimum concrete cover is assumed to be 40 mm.

4.2 Frame's Simulation, loading and analyses

The three RC frames mentioned above are modeled, loaded and analyzed using OpenSEES (2012) as open source object-oriented software. The "*ElasticBeamColumn*" element is used for simulation of beams and columns. To account for the effect of cracking, the moment of inertia of the cross-section for each element is calculated by using the following equation (ACI318, 2011)

$$I_{cr}:\begin{cases} {I_{cr}}^{b} = 0.35{I_{gr}}^{b} \\ {I_{cr}}^{c} = 0.70{I_{gr}}^{c} \end{cases} \tag{20}$$

where ${I_{cr}}^{b}$, ${I_{cr}}^{c}$, ${I_{gr}}^{b}$ and ${I_{gr}}^{c}$ are the cracked moment of inertia of the section of the beams and columns, the gross moment of inertia of the section of the beams and columns, respectively. The ACI318 (2011) code provides the modulus of elasticity of the concrete as $E_c = 4700\sqrt{f_c'}$ in MPa. Here, the modules of elasticity of concrete E_c and steel bars E_s are respectively assumed to be 24870 and 210000 MPa; $f_c^{'}$ and f_y are considered 20 and 414 MPa, respectively; ε_{cu} is assumed to be 0.003; and the weight per unit volume of steel bars is considered as 7.697×10^{-5} N/mm^3.

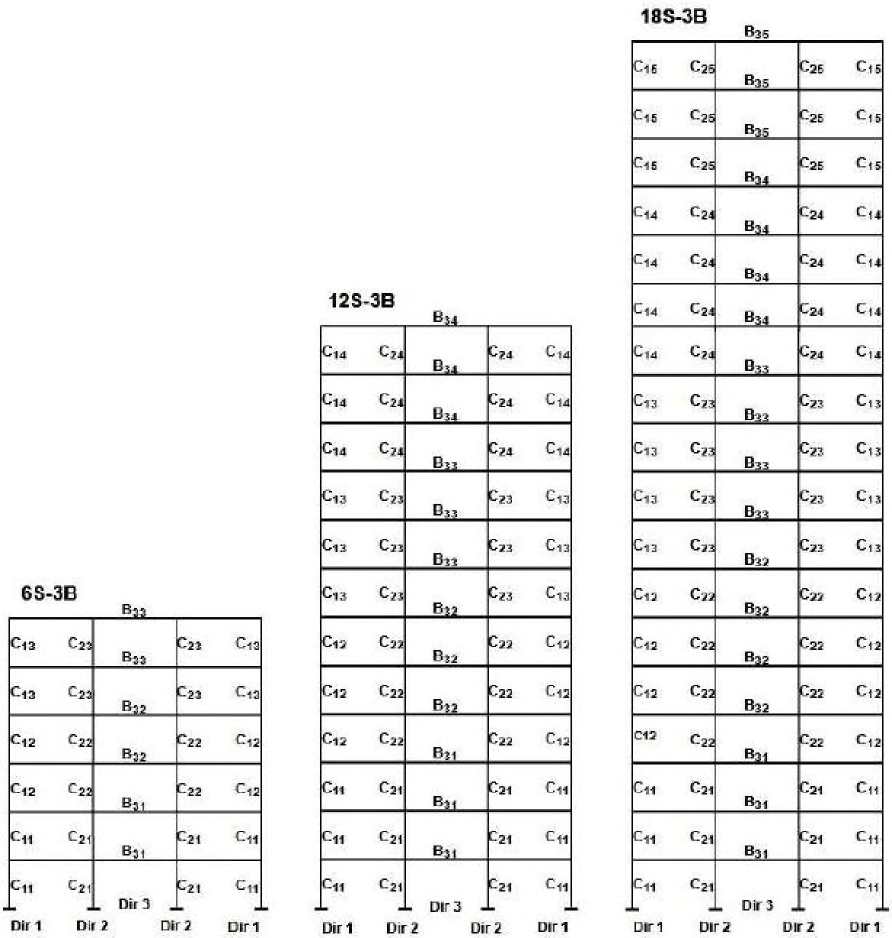


Fig. 4 TCM classification for RC multi-storey frame structures considered

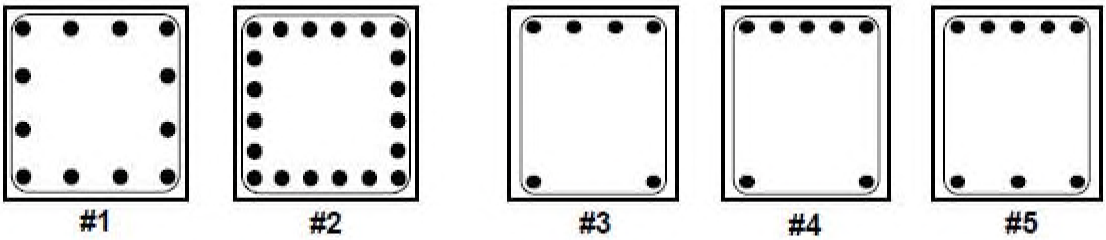


Fig. 5 Arrangements of steel bars in cross-sections of RC frame structures

The applied loads are taken to cover the required load combinations provided by ACI318 (2011) and IBC (2012). Four basic load combinations (*comb*1 to *comb*4) are considered as

$$\begin{aligned} &comb1: 1.4D \\ &comb2: 1.2D + 1.6L \\ &comb3: (1.2 + 0.2S_{DS})D + \mu L \pm \rho E \\ &comb4: (0.9 - 0.2S_{DS})D + \mu L \pm \rho E \end{aligned} \tag{21}$$

in which *D*, *L*and *E*, are the dead, live and earthquake loads acting on the RC frames, respectively. In this study, the values of the uniformly distributed dead and live loads are takenas5.884 kN/m^2 and 1.961 kN/m^2 for all storeys and 6.374 kN/m^2 and 1.471kN/m^2 for the roof level. The self-weight of the frame is automatically added to the dead loads. The weight of walls is not considered here but can be easily included. The redundancy parameter ρ which accounts for redundancy in the structure depending on the seismic design category is taken to range from 1.0 to 1.3. The parameter μ appearing in Eq. (21) above; the ratio of live load equals 0.5 (ACI318, 2011; IBC, 2012). The S_{DS} is the design response acceleration parameter at short period of structure based on IBC (2012) corresponding to the structural site.

The earthquake load (*E*) used in *comb*3 and *comb*4 is computed based on the seismic equivalent static lateral load pattern recommended by (IBC, 2012) as

$$F_i = \frac{w_i h_i^k}{\sum_{j=1}^{ns} w_j h_j^k} V_b \tag{22}$$

where F_i and V_b are the static lateral load at storey level *i* and the base-shear force, respectively; w_i and w_j are the effective weights located or assigned to storey level *i* or *j*; h_i and h_j are the height from the base to the storey level *i* or *j*; *ns* is the number of stories; *k* is an exponent that is determined based on the fundamental period of the structure which is equal to 1 or 2 for structures with a period of 0.5 s or less and for structures having a period of 2.5 s or more, respectively. For structures having a period between 0.5 and 2.5 s, *k* is determined by linear interpolation.

Based on equivalent static lateral force procedure recommended by IBC (2012), seismic equivalent static lateral base shear, V_b^s, is computed from the following equation

$$V_b^s = C_s W_e \tag{23}$$

in which C_s and W_e are the seismic response coefficient and the effective seismic weight in accordance with IBC (2012), respectively. C_s shall be determined based on following equation associated with the given permissible boundaries as follows

$$C_s = \frac{S_{DS}}{(R/I_e)} < \begin{cases} \dfrac{S_{D1}}{T(R/I_e)} & \text{for} \quad T \le T_L \\ \dfrac{S_{D1}T_L}{T^2(R/I_e)} & \text{for} \quad T > T_L \end{cases} \tag{24}$$

where S_{D1} is the design response acceleration parameter at a period of 1s; $R$and I_e are the response modification and importance factors, respectively; T and T_L are the fundamental period of structure and the long-period transition, respectively.

According to IBC (2012), the design storey drift shall be computed as the difference of the deflections at centers of mass at the top and bottom of the storey under consideration. The deflection at storey level i, used to compute design storey drift shall be determined in accordance with the following equation

$$\Delta_i = \frac{C_d\, \Delta_{ie}}{I_e} \tag{25}$$

in which Δ_{ie} is the deflection at the storey level i determined by elastic analysis; and C_d is the deflection amplification factor. R and C_d are considered to be equal to 5.0 and 4.5 in the case of RC frames with intermediate ductility. More information on these parameters can be found in IBC (2012).

In this paper, it is assumed that the frame structures under consideration are constructed at the site with coordinate 31.7598^{o}N (site latitude), 106.4869^{o}W (site longitude) corresponding to the mapped hazard for El Paso, Texas at United States with soil class D and seismic design category “C”. The related seismic parameters are determined from Design Map Detailed Report taken from Web-ground motion parameter calculator from the United States Geological Survey (USGS) (2013). Based on these inputs, the mapped acceleration parameters at short-period (S_S) and 1s period (S_1) are equal to 0.348 and 0.108, respectively. Corresponding to the site class D, the values of site coefficient F_a and F_v, in proportion to S_1 and S_S, are obtained to be 1.522 and 2.368 (using linear interpolation).The maximum considered earthquake spectral response accelerations for short period (S_{MS}), and 1s period (S_{M1}) are obtained to be 0.529g and 0.256g, respectively. This leads to S_{DS}=2/3 × S_{MS} = 0.353g and S_{D1}=2/3 × S_{M1} =0.171g which are the design response acceleration parameters at short and 1s period, respectively. The redundancy factor is taken equal to 1.0 corresponding to the seismic design category C (IBC, 2012). In most of previously presented works, a constant lateral loads were acted at the storey levels for dissimilar structures. In this paper, during the optimization process, in proportion to the changing in W_e and w_i at storey levels, the lateral loads at each storey level are automatically updated for each structural design candidates.

5. Numerical examples

5.1 Optimal design of low- and medium-rise RC frames under static loads

6S-3B and 12S-3BRC frames are considered for optimal design under the combined effect of gravity loads and lateral static loads due to earthquakes. In these frames, beams and columns are classified using the proposed TCM. Equal bays length of 6.0 m and equal storey height of 3.3 m are adopted. The P-Δ effect is automatically considered in the analyses by using a leaning column as depicted in Fig. 6. Thus, when this effect is considered, the design constraints on the slenderness of columns recommended by ACI318 (2011) and the stability criteria of IBC (2012) provision are eliminated. The design optimization of exemplified frame structures is conducted for two design categories including "with" and "without" SCWB concept. The process is performed by means of 25 particles as randomly generated design candidates. Since the PSO may not converge to the optimal solution in a single run, 10 independent optimization processes are implemented by PSO to arrive at the best solution. Subsequently, the optimal run including the least value of objective function is chosen.

After the implementation of optimization procedure schematized in an automated step-by-step process using a link between MATLAB (2010) and OpenSEES (2012), the optimum design candidate of the frame for the above two mentioned categories are obtained. The optimum results of the 6S-3B and 12S-3B frames including the section dimensions (b×h), steel reinforcement ratios (SR) and arrangements, construction cost and structural weight are listed in Tables 1 and 2. The steel ratios of columns and beams for these frames in both categories have been determined to be between 1% and 2%, and less than 1%, respectively. The numerical results summarized in these Tables imply that both construction cost and structural weight are larger when considering SCWB than those of the frames without SCWB. Thus, Table 1 implies that for 6S-3B frame, consideration of SCWB leads to increasing the dimensions (b×h) and steel reinforcement area (A_s) of cross-section of columns particularly in upper stories due to satisfying the SCWB constraint. Therefore, for this frame, the construction cost is about$ 85826 which is 26% higher than without SCWB. In Table 2, similar results are obtained for the 12S-3B frame. The construction cost of this frame with SCWB is about$ 198471 which is about 20% larger than that with SCWB. For both design categories, *comb*2 and *comb*3 are the critical load combinations, respectively.

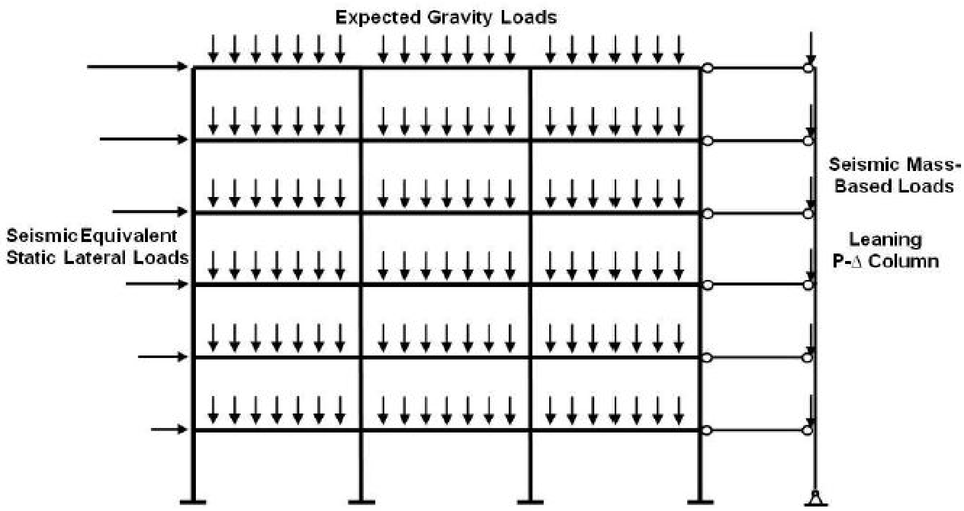


Fig. 6 Structural system with leaning P-Δ column under gravity and lateral static loads

Table 1 Summary of optimum results for 6S-3B optimized RC frame

Element type	Direction number	Group number	Optimum results			
			w/o SCWB		w SCWB	
			b×h (SR)	id-Bar	b×h (SR)	id-Bar
Columns	Dir 1	C_{11}	450×450 (1.19)	#1-D16*	550×550 (1.68)	#2-D18
		C_{12}	450×450 (1.19)	#1-D16	550×550 (1.78)	#2-D14
		C_{13}	450×450 (1.19)	#1-D16	550×550 (1.78)	#2-D14
	Dir 2	C_{21}	550×550 (1.25)	#1-D20	650×650 (1.80)	#2-D22
		C_{22}	400×400 (1.15)	#1-D14	650×650 (1.20)	#2-D18
		C_{23}	400×400 (1.15)	#1-D14	600×600 (1.41)	#2-D18
Beams	Dir 3	B_{31}	350×550 (0.73)	#4-D16	350×550 (0.73)	#4-D16
		B_{32}	350×550 (0.73)	#4-D16	350×550 (0.73)	#4-D16
		B_{33}	350×550 (0.73)	#4-D16	350×550 (0.73)	#4-D16
{Concrete, Steel, Formwork} Cost ($)			{26960, 19286, 18196}		{35383, 29824, 20619}	
Total Construction Cost ($)			64442		85826	
Structural Weight (N)			862238		1137860	

*D16: steel bar of 16 mm diameter and id stands for the identification label of cross-section (see Fig. 5)

Table 2 Summary of optimum results of 12S-3B optimized RC frame

Element type	Direction number	Group number	Optimum results			
			w/o SCWB		w SCWB	
			b×h (SR %)	id-Bar	b×h (SR %)	id-Bar
Columns	Dir 1	C_{11}	600×600 (1.27)	#1-D22	650×650 (1.49)	#2-D18
		C_{12}	550×550 (1.51)	#1-D22	600×600 (1.41)	#2-D18
		C_{13}	500×500 (1.51)	#1-D20	550×550 (1.02)	#2-D16
		C_{14}	450×450 (1.19)	#1-D16	500×500 (1.23)	#2-D16
	Dir 2	C_{21}	650×650 (1.20)	#2-D18	750×750 (1.89)	#2-D22
		C_{22}	600×600 (1.41)	#2-D18	650×650 (2.14)	#2-D22
		C_{23}	550×550 (1.01)	#1-D18	650×650 (1.20)	#2-D18
		C_{24}	450×450 (1.19)	#1-D16	600×600 (1.12)	#2-D18
Beams	Dir 3	B_{31}	450×550 (0.72)	#4-D18	450×550 (0.65)	#5-D16
		B_{32}	450×500 (0.79)	#4-D18	450×550 (0.65)	#5-D16
		B_{33}	450×500 (0.79)	#4-D18	450×550 (0.65)	#5-D16
		B_{34}	450×500 (0.79)	#4-D18	450×550 (0.65)	#5-D16
{Concrete, Steel, Formwork} Cost ($)			{70844, 55593, 41057}		{83515, 70458, 44498}	
Total Construction Cost ($)			167494		198471	
Structural Weight (N)			2272562		2685795	

As one of the effective design constraints in the optimum design process, the demand to capacity ratio (DCR) of beam and column elements are investigated herein. The dominant DCRs of the structural elements of each class in different directions of 1, 2 and 3 for 6S-3B and 12S-3B optimized frames is compared and shown in Fig. 7. As shown in this figure, the ratio is given for both categories including design optimization with and without considering SCWB constraint. The average dominant DCRs of columns for the 6S-3B optimized frame with SCWB is about 0.70 while the value is about 0.40 for that of the frame without SCWB. The average dominant DCR of beams for the frame with SCWB is higher than 0.90 for both frames with and without SCWB. For the 12S-3B optimized frame with SCWB, the average dominant DCR of columns and beams were about 0.50 and 0.86, respectively; while they are about 0.65 and 0.98 for the frame without SCWB. The SCWB ratio of optimally designed 6S-3B and 12S-3B frames for both mentioned categories are shown in Fig. 8. For 6S-3B and 12S-3B optimized frames with SCWB, the SR decreases along the height of the frame and is almost constant for internal joints (direction 2 as “Dir 2”) and external joints (direction 1 as “Dir 1”). When the SCWB constraint is omitted, the difference between the joints at first storey and top storey is less than those of the case with SCWB. Also, for the frames without SCWB, there is a significant difference between SCWB ratio of joints at direction 1 and 2. Some of these differences could be due to the role of other design constraints.

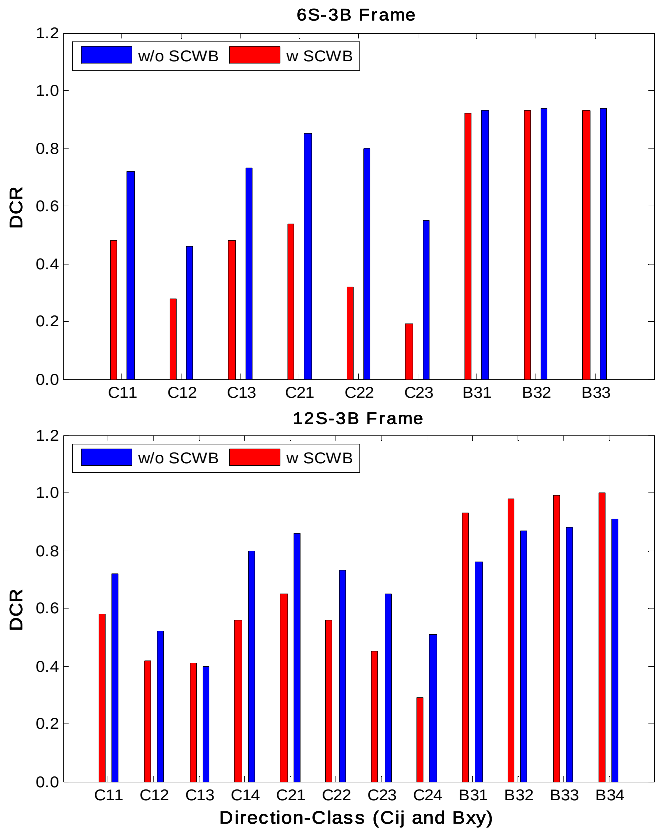


Fig. 7 The dominant DCRs of 6S-3B and 12S-3B optimized frames with and without SCWB constraint based on identification number of direction-class of elements

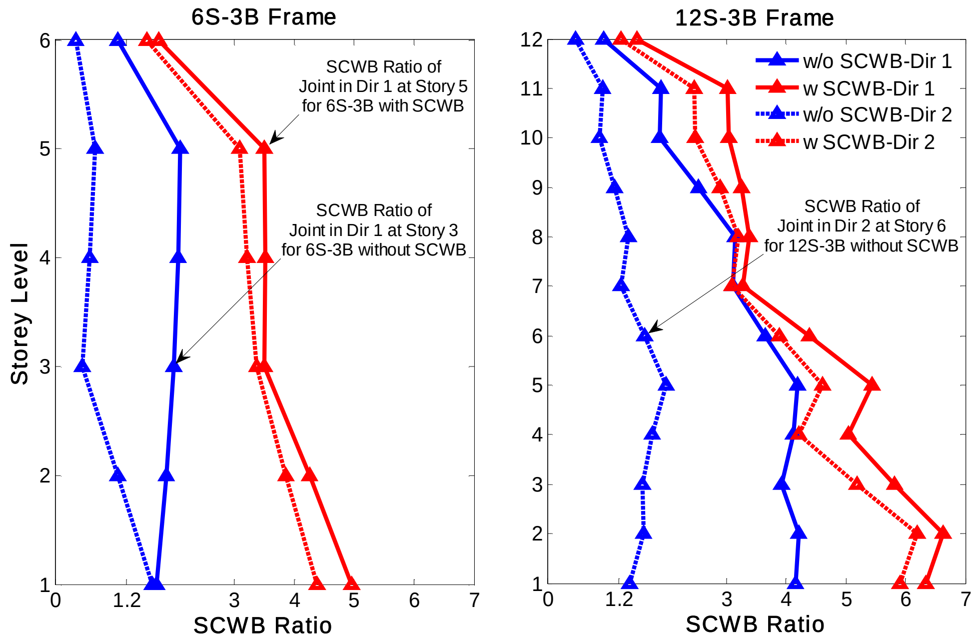


Fig. 8 SCWB ratio of 6S-3B and 12S-3B optimized RC frames with and without SCWB constraint

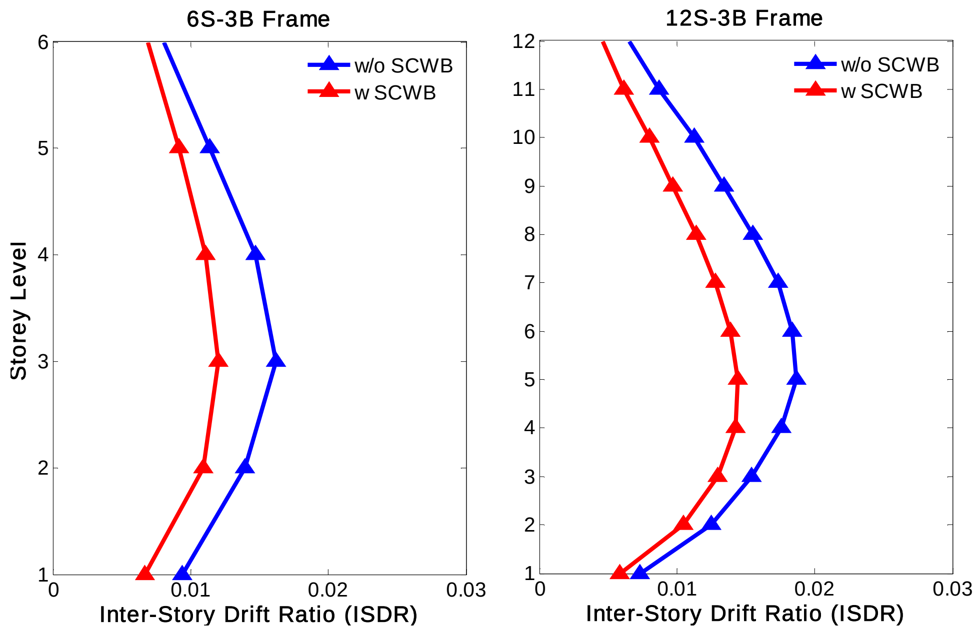


Fig. 9 ISDR for optimized 6S-3B and 12S-3B RC frames with and without SCWB constraint

Concerning the ISDR criteria as shown in Fig. 9, for the 6S-3B and 12S-3B optimized frames in both abovementioned categories; the related ratio is located in the allowable range that is less than 0.025. As depicted in the figure considering the SCWB constraint in the design optimization process leads to reduction in the ISDR over the height of the frames.

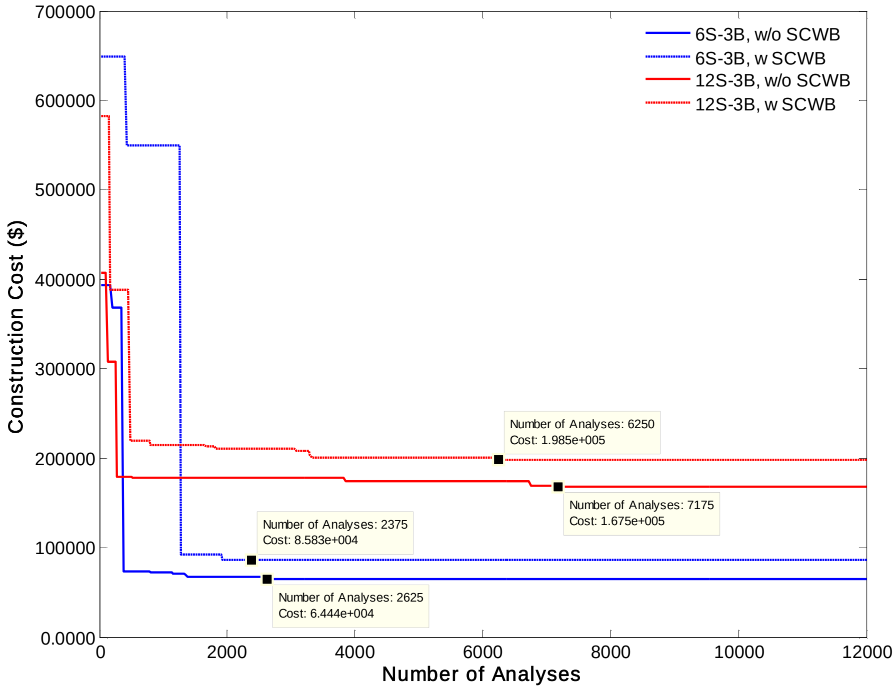


Fig. 10 Convergence history of objective function and necessary number of iterations to achieve the optimum solution for 6S-3B and 12S-3B optimized RC frames with and without SCWB constraint

The number of analyses to reach the optimum solution for the 6S-3B and 12S-3B optimized frames is examined next (see Fig. 10). By using the TCM technique, the first set of constraints is eliminated from the optimization process. Also, the number of real-design variables reduces from nine to three pseudo-design variables for the 6S-3B frame, and from twelve to three for the 12S-3B frame. Indeed, the number of analyses to reach the optimum design solution is significantly less than that of presented in the literature listed in Section 2 (e.g. Gholizadeh and Aligholizadeh, 2013). For the 6S-3B frame, convergence to optimal solution is achieved after2375 and 2625 analyses with and without SCWB, respectively. Also, the number of analyses to reach optimum point is obtained to be 6250 and 7175 for 12S-3B with and without SCWB, respectively. This significant reduction is due to the reduction in the number of design variables and constraints associated with smart pre-defined database. This, in turn, indicates the effectiveness of the proposed TCM.

5.2 Optimal design of high-rise RC frames under static and time-history lateral loads

Nowadays, structural optimization has received great attention among researchers, and engineers of high-rise building structures. Design optimization of high-rise frames, as large-scale structures, is a rather difficult task (Papadrakakis *et al*. 2000, Papadrakakis *et al*. 2001, Papadrakakis *et al*. 2002, Gholizadeh and Fattahi 2013). In seismic design of high-rise buildings, the time-history analysis is the most accurate method of analysis compared to the response spectrum method and the equivalent static load method. Based on IBC (2012) code provisions, at least three ground motion records compatible with the structural site can be used for seismic analysis and design. The records shall be scaled such that the average value of the 5 percent damped response spectra for the suite of ground motions is not less than the design response spectrum for the site for periods ranging from 0.2*T* to 1.5*T* (*T* is the fundamental natural period of the structure) for the direction of response being analyzed. Also, based on IBC (2012), where the maximum scaled base-shear predicted by time-history dynamic analysis V_b^d is less than 85 % of the value of base-shear determined by Eq. (23).The scaled member forces and drifts shall be additionally multiplied by normalizing factor of (V_b^s/V_b^d) and 0.85(V_b^s/V_b^d), respectively. In the optimization process for this frame, the third set of constraints stated in section 2.2.3 shall be additionally considered. The maximum time-dependent response of three used records shall be used in design (IBC 2012).

This section presents the design optimization of the 18S-3B RC frame for two loading types, namely, 1) combination of gravity and seismic equivalent static lateral loads, 2) combination of gravity and time-history earthquake loads. The length of beams and height of columns are taken the same as in the previous examples. As shown in Fig. 4, the beams and columns are separately classified using the proposed TCM method. As shown in Fig. 11, three artificial ground motion records compatible with the response spectrum corresponding to the mapped hazard for El Paso, Texas are treated as ground motion excitations to the frame structure. Three envelope functions including Jennings-compound, Housner-Trapezoidal and Trigonometric functions (Jennings *et al.* 1968, SeismoArtif 2013) available in the SeismoArtif program (2013) are used to create these artificial records. The envelope parameters are selected such that the total duration is about 20 s, and the rise and level time are selected to be 5s and 15s, respectively. This frame is analyzed under gravity and equivalent static loads (i.e. *comb*1 and *comb*2) and also under gravity and ground acceleration loads (*comb*3 and *comb*4). Then, the Eqs. (14), (15) and (16) are checked to be satisfied. The Newmark-β method is employed in the numerical integration of the equations of motion. A 5% Rayleigh damping model is adopted. During the optimization procedure, the optimal solution leads to different cross-sections of the structural members and hence the natural frequencies for each case are also different. Thus, a free vibration analysis is automatically carried out for each design candidate to compute the coefficients of Rayleigh relationship.

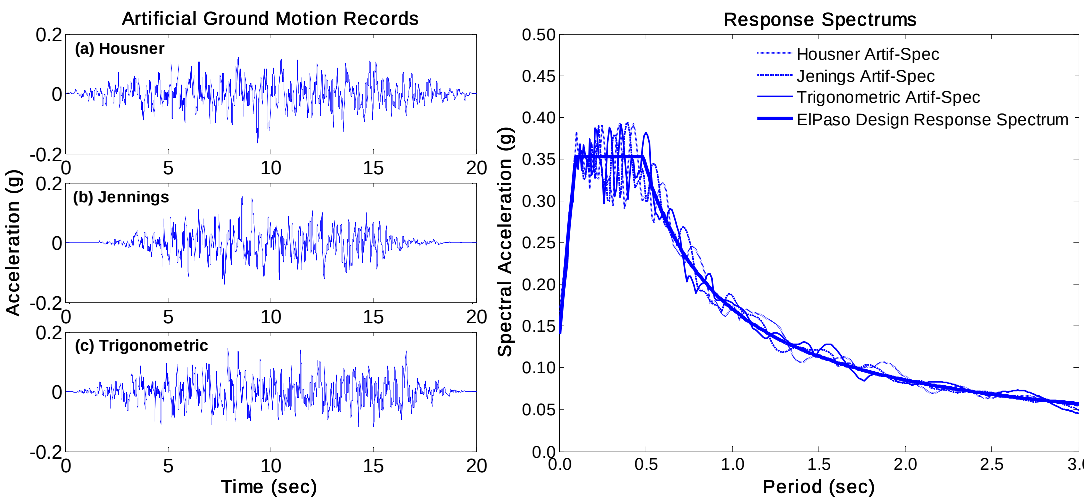


Fig. 11 Artificially simulated earthquake records and associated design response spectrum generated from them

Based on IBC (2012), in order to use the earthquake records, we need to compute *T* of each design candidate (particle) during the optimization process. Since the parameter changes during the process, for each particle, an individual scaling/matching process needs to be conducted, increasing the computational burden. Hence, by considering a reasonable assumption, once scaling/matching process is conducted before starting the optimization process.

After implementing the optimization procedure for the two mentioned loading types, the optimum results of the high-rise frame were obtained. These results include *b* and *h*, SR and arrangements, construction cost and structural weight (see Table 3). The optimal SR for cross-sections of columns for the 18S-3B frames under load types (1) and (2), was found to be 1 and 3%, and about 1 and 2%, respectively. For both loading types, the SR for beams cross-sections was less than about 1%. Table 3 implies that the construction cost of the 12S-3B optimized frame under load types of (1) and (2) is about$ 332006 and about $ 322857, respectively. The dominant DCRs of the structural elements of each class in directions 1, 2 and 3 for 18S-3B optimized frame for both loading types are estimated (see Fig. 12). Thus, under load type (1), the average dominant DCR of columns and beams were about 0.55 and 0.90, respectively while the associated value is about 0.62 and 0.89, respectively for the same frame structure under load type (2). It was seen that for the two loading types, *comb*2 and *comb*3 are the critical load combinations.

Table 3 Summary of optimum results for 18S-3B optimized RC frame

Element type	Direction number	Group number	Optimum results (with SCWB)			
			Seismic Equivalent static		Time-history earthquake	
			b×h (SR %)	id-Bar	b×h (SR %)	id-Bar
Columns	Dir 1	C_{11}	800×800 (1.92)	#2-D28	650×650 (1.80)	#2-D22
		C_{12}	550×550 (2.99)	#2-D24	550×550 (1.68)	#2-D18
		C_{13}	550×550 (2.99)	#2-D24	550×550 (1.68)	#2-D18
		C_{14}	550×550 (1.33)	#2-D16	500×500 (2.04)	#2-D18
		C_{15}	550×550 (1.33)	#2-D16	500×500 (1.23)	#2-D14
	Dir 2	C_{21}	800×800 (0.98)	#2-D20	800×800 (1.66)	#2-D26
		C_{22}	750×750 (1.12)	#2-D20	750×750 (1.35)	#2-D22
		C_{23}	750×750 (1.12)	#2-D20	750×750 (1.35)	#2-D22
		C_{24}	750×750 (1.12)	#2-D20	600×600 (2.11)	#2-D22
		C_{25}	700×700 (1.28)	#2-D20	600×600 (2.11)	#2-D22
Beams	Dir 3	B_{31}	450×600 (0.52)	#4-D16	450×650 (0.61)	#4-D18
		B_{32}	450×600 (0.52)	#4-D16	450×650 (0.61)	#4-D18
		B_{33}	450×600 (0.52)	#4-D16	450×600 (0.66)	#4-D18
		B_{34}	450×600 (0.52)	#4-D16	450×600 (0.66)	#4-D18
		B_{35}	450×600 (0.52)	#4-D16	450×550 (0.72)	#4-D18
{Concrete, Steel, Formwork} Cost ($)			{143261, 117574, 71170}		{133650, 120072, 69135}	
Total Construction Cost ($)			332006		322857	
Structural Weight (N)			4602695		4308244	

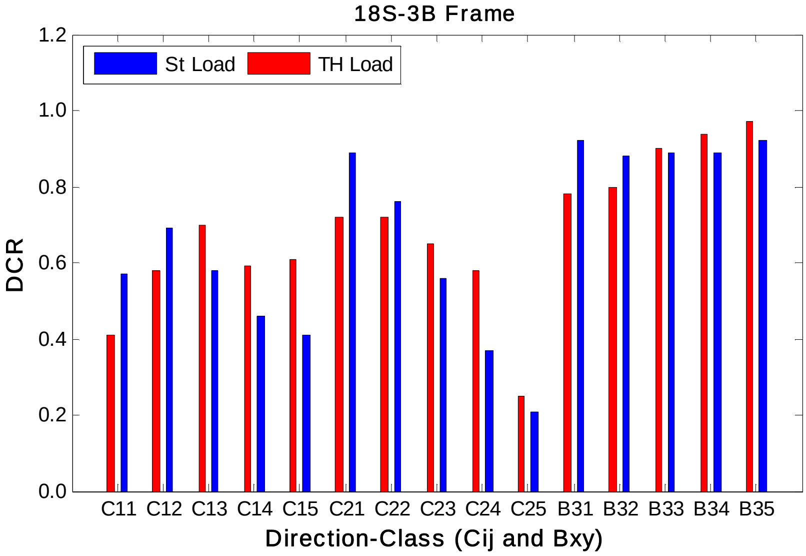


Fig. 12 Dominant DCRs of 18S-3B optimized RC frames for static and earthquake loads based on identification number of direction-class of elements

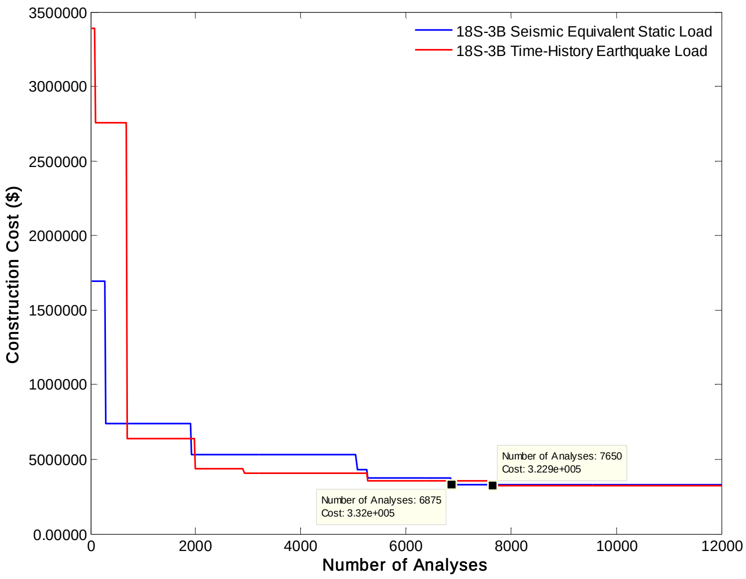


Fig. 13 Convergence history of objective function and necessary number of iterations to achieve the optimum solution for 18S-3B optimized RC frame under static and earthquake loads

Additional to reducing the number of constraints, the TCM reduces also the number of real design variables to 3 pseudo-design variables in current work compared to a significantly larger number of design variables, see, e.g. (Gholizadeh and Aligholizadeh, 2013). Therefore, the number of analyses to find optimum solution for the 18S-3B optimized frames depicted in Fig. 13 was found to be 6875 and 7650 for load types (1) and (2), respectively. These values are significantly less than those obtained in similar reported works (Gholizadeh and Aligholizadeh, 2013).

6. Conclusions

This paper proposes a new, efficient and automated optimization methodology, using Tree Classification Method (TCM), for optimal seismic design of RC structures. The advantage of the TCM is the reduction of the number of design variables and constraints. This leads to significant reduction in the computational cost since the optimal solution is achieved faster. Additionally, the design constraints on the cross-sections and other practical restrictions are considered in a pre-processing step that precedes the main optimization process. For instant, in the 6S-3B frame, the number of design variables reduces from 9 real variables to 3 pseudo-design variables. The same quantity reduces from 12 and 15 real variables to 3 pseudo-design variables, respectively for the 12S-3B and 18S-3B frame structures.

The constraints considered in optimal design of frame structures under gravity and earthquake loads conform to the ACI318 (2011) and IBC (2012). The strong column-weak beam (SCWB) concept is also considered in the optimal design of RC frame structures under the combined effect of static and earthquake loads. The resulting optimal solution considering the SCWB concept is seen to be an active constraint for columns. The SCWB constraint yields stronger columns compared to the case without considering the SCWB constraint. Also, the ground acceleration causes a decreasing in the construction cost, structural weight and corresponding base shear compared with the case of seismic equivalent static loads. The following important points can be concluded:

• The proposed TCM leads to a significant decrease in the number of design constraints and changes the role of real design variables as pseudo-design variables in the optimization process resulting in small number of analyses to find the optimum design candidate;

• By using the TCM, the wide range of dimensions and steel bar diameters associated with numerous arrangements can be easily used to optimum design with respect to similar works reported in literature;

• Additional to decreasing the length of position and velocity vectors of real valued model of PSO algorithm compared with its binary model, utilizing the TCM leads to significant decrease in the length of these vectors again leading to a faster convergence rate to the optimal solution;

• The TCM can simplify the design optimization of large-scale and complex structural systems under dynamic loads such as earthquake and wind loads;

• The SCWB constraint has a significant effect on the engineering demand parameters such as DCRs and ISDR, as well as construction cost and structural weight;

The proposed TCM technique is useful for high time-consuming structural optimization problems such as design optimization of large scale 2D and 3D structural systems; performance-based optimum seismic design of structures considering the nonlinear static and dynamic analyses and for optimal design of structures considering soil-structure interaction. In this paper, the TCM technique was explained for optimal design of RC frame structures but can be applied to other structures of different construction materials such as steel trusses, RC shear walls and composed large-span bridges. Future research could be extended to optimal design of structures under variable critical earthquake loads for sites having limited earthquake data. Herein, additional constraints on the optimal earthquake input (Moustafa 2011, Takewaki *et al.* 2013) will be also included beside the original constraints on the cross-sections and material capacities. A large number of analyses to find the global optimum solution are clearly needed.TCM could be also integrated with stochastic analysis to handle optimal design of structures considering uncertainty in loads and variability in structural properties.

As a recommendation, these benchmark code-based optimized RC frames may be utilized to comparison of optimization algorithms capability, as well as seismic performance evaluation of code-based seismic designed RC frame structures.

References

ACI Committee 318 (2011), *Building code requirements for structural concrete and commentary*, Detroit.

Balling, R.J. and Yao, X. (1997), “Optimization of reinforced concrete frames”, *J. Struct. Eng.*, **123**(2), 193-202.

Bergh, V. and Engelbrecht, A. (2003), “Using neighborhood with the guaranteed convergence PSO”, *Proceedings of the 2003 IEEE swarm intelligence symposium*, 235-242.

Camp, C.V. and Huq, F. (2013), “CO2 and cost optimization of reinforced concrete frames using a big bang-big crunch algorithm”, *Eng. Struct.*, **48**, 363-372.

Camp, C.V., Pezeshk, S. and Hansson, H. (2003), “Flexural design of reinforced concrete frames using a genetic algorithm”, *J. Struct. Eng.*, **129**(1), 105-115.

Fadaee, M.J. and Grierson D.E. (1996), “Design optimization of 3D reinforced concrete structures”, *Struct. Multidiscip. Opt.*, **12**, 127-34.

Fadaee, M.J. and Grierson, D.E. (1998), “Design optimization of 3D reinforced concrete structures having shear walls”, *Eng. Comput.*, **14**(2), 139-145.

Fragiadakis, M. and Lagaros, N.D. (2011), “An overview to structural seismic design optimisation frameworks”, *Comput. Struct.*, **89**(11), 1155-1165.

Fragiadakis, M. and Papadrakakis, M. (2008), “Performance-based optimum seismic design of reinforced concrete structures”, *Earthq. Eng. Struct. Dy.*, **37**(6), 825-844.

Gerlein, M.A. and Beaufait, F.W. (1980), “An optimum preliminary strength design of reinforced concrete frames”, *Comput. Struct.*, **11**(6), 515-524.

Gharehbaghi, S. and Fadaee, M.J. (2012), “Design optimization of RC frames under earthquake loads”, *Int. J. Opt. Civil Eng.*, **2**, 459-477.

Gharehbaghi, S. and Khatibinia, M. (2015), “Optimal seismic design of reinforced concrete structures under time-history earthquake loads using an intelligent hybrid algorithm”, *Earthq. Eng. Eng. Vib.*, **14**(1), 97-109.

Gharehbaghi, S., Salajegheh, E. and Khatibinia, M. (2011), “Optimization of reinforced concrete moment resistant frames based on uniform hysteretic energy distribution”, *Proceedings of the 1stinternational conference on urban construction in the vicinity of active faults*, Tabriz, Iran.

Gharehbaghi, S., Salajegheh, E. and Khatibinia, M. (2012), “Evaluation of seismic energy demand of reinforced concrete moment resistant frames considering soil-structure interaction effects”, *Proceeding of the eleventh international conference on computational structures technology*, B.H.V. Topping (ed.), Civil-Comp Press, Dubrovnik, Croatia.

Gholizadeh, S. (2013), “Layout optimization of truss structures by hybridizing cellular automata and particle swarm optimization”, *Comput. Struct*, 125, 86-99.

Gholizadeh, S. and Aligholizadeh, V. (2013), “Optimum design of reinforced concrete frames using BAT meta-heuristic algorithm”, *Int. J. Opt. Civil Eng.*, **3**(3), 483-497.

Gholizadeh, S. and Fattahi, F. (2014), “Design optimization of tall steel buildings by a modified particle swarm algorithm”, *Struct. Des. Tall Special Build.*, **23**(4), 285-301.

Gholizadeh, S. and Salajegheh, E. (2009), “Optimal design of structures for time history loading by Swarm intelligence and an advanced metamodel”, *Comput. Meth. App. Mech. Eng*., **198**, 2936-2949.

Goldberg, D.E. (1989), *Genetic algorithms in search, optimisation, and machine learning*, Reading MA: Addison-Wesley.

Guerra, A. and Kiousis P.D. (2006), “Design optimization of reinforced concrete structures”, *Comput. Concrete*, **3**(5), 313-334.

IBC (2012), *International Building Code, International Code Council*, Country Club Hills, USA.
Jennings, P.C., Housner, G.W. and Tsai, N.C. (1968), "Simulated earthquake motions", EERL Report, California Institute of Technology.
Kanno, Y. and Takewaki, I. (2007), "Global optimization algorithm for worst-case limit analysis of trusses under load uncertainties", *Proceedings of the 4th international conference on computational methods, ICCM*, April 4-6, Hiroshima, Japan.
Kao, C.H. and Yeh, I.C. (2014), "Optimal design of reinforced concrete plane frames using artificial neural networks", *Comput. Concrete*, **14**(4), 445-462.
Kaveh, A. and Behnam, A.F. (2013), "Design optimization of reinforced concrete 3D structures considering frequency constraints via a charged system search", Sci. Iranica, **20**(3), 387-396.
Kaveh, A. and Sabzi, O. (2011), "A comparative study of two meta-heuristic algorithms for optimum design of reinforced concrete frames", *Int. J. Civil Eng.*, **9**(3), 193-206.
Kaveh, A. and Sabzi, O. (2012), "Optimal design of reinforced concrete frames using big bang-big crunch algorithm", *Int. J. Civil Eng.*, **10**(3), 189-200.
Kaveh, A. and Zakian, P. (2014a), "Seismic design optimization of RC moment frames and dual shear wall-frame structures via CSS algorithm", *Asian J. Civil Eng.*, 15(3), 435-465.
Kaveh, A. and Zakian, P. (2014), "Optimal seismic design of Reinforced Concrete shear wall-frame structures", *KSCE J. Civil Eng.*, **18**(7), 2181-2190.
Kennedy, J. and Eberhart, R.C. (2001), *Swarm intelligence*, Academic Press, San Diego, California.
Khatibinia, M., Gharehbaghi, S. and Moustafa, A. (2015), *Seismic reliability-based design optimization of reinforced concrete structures including soil-structure interaction effects*, Earthquake Engineering-From Engineering Seismology to Optimal Seismic Design of Engineering, in: Moustafa, A. (ed.), InTech, Croatia, 267-304.
Krishnamoorthy, C.S. and Munro, J. (1973), "Linear program for optimal design of reinforced concrete frames", *Proceedings of IABSE*, **3**, 119-141.
Kwak, H.G. and Kim, J. (2009), "An integrated genetic algorithm complemented with direct search for optimum design of RC frames", *Comput.Aid. Des.*, **41**(7), 490-500.
Lagaros, N.D. (2014), "A general purpose real-world structural design optimization computing platform", *Struct. Multidisciplin. Opt.*, **49**(6), 1047-1066.
Lagaros, N.D. and Papadrakakis, M. (2007), "Seismic design of RC structures: A critical assessment in the framework of multi-objective optimization", *Earthq. Eng. Struct. Dy.*, **36**(12), 1623-1639.
Lee, C. and Ahn, J. (2003), "Flexural design of reinforced concrete frames by genetic algorithm", *J. Struct. Eng.*, **129**(6), 762-74.
MATLAB (2010), *The language of technical computing*, Math Works Inc.
Moharrami, H. and Grierson, D.E. (1993), "Computer automated design of reinforced concrete frameworks", *J. Struct. Eng.*, **119**(7), 2036-2058.
Moustafa, A. (2011), "Damage-based design earthquake loads for SDOF inelastic structures", *J. Struct. Eng.*, **137**(3), 456-467.
Moustafa, A. (2013), *Earthquake-resistant structures: design, assessment and rehabilitation*, InTech, Croatia.
Moustafa, A. (2015), *Earthquake engineering- from engineering seismology to optimal seismic design of engineering*, InTech, Croatia.
OpenSEES (2012), "Open system for earthquake engineering simulation", Pacific Earthquake Engineering Research Centre, University of California, Berkeley, (Release 2.3.2).
Papadrakakis, M., Lagaros, N.D. and Plevris, V. (2001a), "Optimum design of space frames under seismic loading", *Int. J. Struct. Stab. Dy.*, **1**(1), 105-124.
Papadrakakis, M., Lagaros, N.D., Tsompanakis, Y. and Plevris, V. (2001), "Large scale structural optimization: computational methods and optimization algorithms", *Arch. Comput. Method. Eng.*, **8**(3), 239-301.
Papadrakakis, M., Lagaros, N.D. and Plevris V. (2000), "Optimum design of structures under seismic loading", *Proceeding of the European congress on computational methods in applied sciences and*

engineering, Barcelona, Spain.
Papadrakakis, M., Lagaros, N.D. and Plevris, V. (2002), “Multi-objective optimization of skeletal structures under static and seismic loading conditions”, *Eng. Opt.*, **34**(6), 645-669.
Plevris, V., Mitropoulou, C.C. and Lagaros, N.D. (2012), *Structural seismic design optimization and earthquake engineering: formulations and applications*, IGI Global.
Rajeev, S. and Krishnamoorthy, C.S. (1989), “Genetic algorithm-based methodology for design optimization of reinforced concrete frames”, *Comput. Aid. Civil Infrastruct. Eng.*, **13**(1), 63-74.
Salajegheh, E., Gholizadeh, S. and Khatibinia, M. (2008), “Optimal design of structures for earthquake loads by a hybrid RBF-BPSO method”, *Earthq. Eng. Eng. Vib.*, 7(1), 14-24.
Seismosoft Programs (2013), SeismoArtif, http://www.seismosoft.com.
Seyedpoor, S.M., Salajegheh, J., Salajegheh, E. and Gholizadeh, S. (2011), “Optimal design of arch dams subjected to earthquake loading by a combination of simultaneous perturbation stochastic approximation and particle swarm algorithms”, *Appl. Soft Comput.*, **11**(1), 39-48.
Shi, Y. and Eberhart, R.C. (1997), “A modified particle swarm optimizer”, *Proceedings of the IEEE international conference on evolutionary computation*, 303-308.
Takewaki, I., Fujita, K. and Moustafa, A. (2013), *Improving the earthquake resilience of buildings: the worst case approach*, Springer Series in Reliability Engineering, London.
Web-Ground Motion Parameter Calculator. United States Geological Survey (USGS) (2013), Available from: http://earthquake.usgs.gov/hazards/designmaps/javacalc.php.
Zou, X.K. and Chan, C.M. (2005), “An optimal resizing technique for seismic drift design of concrete buildings subjected to response spectrum and time history loadings”, *Comput. Struct.*, **83**(19), 1689-1704.

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