Detailed optimum design of reinforced concrete frame structures

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Abstract: - This paper presents an optimum design approach for reinforced concrete (RC) frame structures which are excited by static and dynamic forces. In this approach, dimensions and reinforcements of all structural members are individually optimized without grouping them for optimum design. During optimization, the design rules given in ACI-318 (Building code requirements for reinforced concrete structures) are considered. Detailed seismic analyses are done by conducting time history analyses. The aim of the optimization is to find the most economical design of the frame structure. Additionally, the weight of the structure is indirectly reduced because of the consideration of dynamic analyses. A modified Harmony Search (HS) algorithm with additional random search iterations is developed for the optimization. The proposed method was demonstrated with a symmetric structure. The results show that the approach is feasible to find optimum designs of different structural members of RC frames.

Key-Words: Optimization, Metaheuristic methods, Reinforced Concrete Structure, Time history analyses, Frame Structures, Harmony Search.

1 Introduction

The optimum design of structures is an important and challenging subject. Especially in design of reinforced concrete (RC) structure, the existing of two different materials which are different cost and behaviour, is the reason of using optimization techniques.

In the optimum design of RC structures or a specific member of a RC structure, metaheuristic methods are very suitable. For that reason, several approaches have been proposed but researches must continue to investigate in order to find detailed and practical optimum design methodologies. The employed metaheuristic algorithms are genetic algorithm [1-4], simulating annealing [5-6], big bang-big crunch [7-9], charged system search [10] and harmony search [11-12].

Akin and Saka [11] employed the HS algorithm for cost optimization of RC plane frame structure by using lateral equivalent static earthquake loads. The column and beam members are grouped and predefined variable pools for cross-section dimensions and reinforcements. These variable pools may be also effective on practical design, but the precise optimum results can be only found by using different sizes of reinforcements. Especially, grouping of columns of different stories are not an effective optimum design since the columns carry out the existing storey force in addition to upper storey forces. Also, dynamic earthquake loads can be considered for a realistic design optimization. By using a classical algorithm, these details cannot be considered since the number of the design variables are too many. Only, particular optimum designs can be found when the other members have a design far from the optimum one or a design with constraint violation. Thus, the classical algorithms can be enriched with additional modifications.

In this study, RC frame structures which are excited by static and dynamic forces are optimized by using a modified harmony search (HS) algorithm with additional random search iterations. Differently from the previous approaches for RC frames, earthquake effects are considered according to time history analyses of three earthquake records. These time history analyses which are updated according to cross-section dimensions were conducted for all iterations of the optimization process. Thus, the earthquake effects are distributed to all joints. Due to the rotational terms of mass, stiffness and damping matrices, earthquakes have effect as moment on the joints because of angular acceleration. These moments are neglected in the previous studies using equivalent lateral earthquake loadings. For static loads, all unfavorable conditions of live loads are considered. Also, beam and column elements are not grouped and reinforcement design properties are not taken from a prepared template. For that reason, the proposed method is free to find the best optimum results. In the optimization process, a detailed reinforcement design considering positioning of the bars is done. For all iteration steps of the optimization, the designs of the RC elements are checked according to ACI-318 building code requirements for structural concrete [13]. The effectiveness of the proposed method is shown with a multi-bay multi-story symmetric RC frame structure.

2 Methodology

A new optimization methodology using additional random search iterations for the optimization of RC frames was generated. The methodology uses additional random search stages for optimization of several design variables and these random stages are combined with employing a music-inspired metaheuristic algorithm called harmony search. Harmony search algorithm (HS), which was developed by Geem et al. [14], is a memory based random search method imitating the process of musical performances. A musician tries to find a perfect state of the harmony in musical performances. Similarly, a global solution is searched in optimization.

Metaheuristic algorithms are developed by the inspiration of several observations and processes in life. Genetic algorithm (GA) is one of the most popular metaheuristic algorithm which is successfully used in several optimization approaches [15, 16]. Recently, GA has been employed in highway alignment optimization [17], resource utilization [18], a computerized feature selection [19], developing crew allocation system [20], dynamic identification of structural systems [21], solving traffic signal coordination [22], dependability assurance in the design of bridges [23], semiconductor hookup construction [24], identification of a smart polymeric textile [25] and optimization of earthquake energy dissipation system [26].

Recently in structural engineering, HS was employed in the optimization of the problems such as cellular beams [27], trusses [28-29], tuned mass dampers [30-32], RC frames [11], structural frames [33], selecting and scaling of ground motion records [34], T-shaped beams [12] and base isolation systems [35].

Due to optimization of various types of structural members (columns and beams constructing RC frames) with different design constraints and different materials such as steel and concrete, a classical metaheuristic algorithm may not be sufficient to reach optimum solution. In optimization approaches using metaheuristic algorithms, the optimum cross sectional dimensions may be combined with random reinforcement designs which are not optimums. If several limitations and grouping of design variables ensuring design constraints are done, classical metaheuristic methods may be effective for a particular optimum solution. For a general optimization, sub-optimization stages can be used. Thus, the proposed methodology uses a modified metaheuristic algorithm which uses additional random search stages during the generation of possible design variables.

The methodology was coded by using Matlab [36]. Thus, time history analyses are conducted with the help of Simulink for all iterations of the optimization process. The methodology is described in this section.

Step 1: First, properties of the structural model are defined. These properties are number of bays, number of stories, number of joints, boundary conditions of joints, coordinates of elements and the joints. Also, Live (L) and Dead (D) distributed loads (except self-weight of structural members) are defined for all spans. These loads can be defined as equally distributed, triangular distributed or trapezium distributed as seen in Fig. 1. In that case, the loads transferred from the slabs can be considered in the optimization. Self-weight of the elements are also calculated according to random cross-sectional dimensions and combined with distributed static loads. Ranges of cross-section dimensions (breadth; b_w and height; h), longitudinal and lateral reinforcements sizes are determined. Design constants such as clear cover (c_c), maximum aggregate diameter (D_{max}), compressive strength of concrete (f'_c), yield strength of steel (f_y), elasticity modulus of steel (E_s), specific gravity of steel (γ_s), specific gravity of concrete (γ_c), the cost of concrete per m^3 (C_c) and the cost of steel per ton (C_s) are defined.

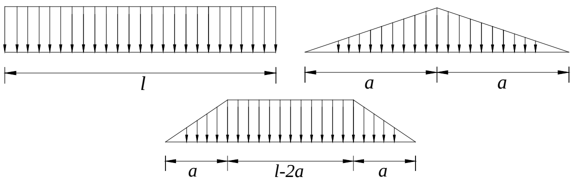

Figure 1 Equally distributed, triangular distributed and trapezium distributed loads

Step 2: After the definition of ranges of design variables and design constants, the initial harmony matrix which contains randomly assigned design variables is generated. This matrix is constructed with harmony vectors. The proposed harmony search algorithm with additional random search iterations contain several randomization stages with different objectives during a harmony vector is constructed. The second step is explained as sub-steps.

Step 2a: First, cross-sectional dimensions of the beams are randomized. Randomizing cross-section dimensions are checked for following statements given in ACI 318.

$$d < \frac{l}{4} \quad (1)$$

$$b_w \geq 0.3h \;. \quad (2)$$

In Eq. (1), depth (d) of the beam is limited to 0.25 times of clear length of the beam (*l*). In the control of these design constraints, the depth of the beam is assumed as 50 mm less than the height of the beam. After the design of the reinforcements, the exact value of d is calculated and calculations are updated. Then, all constraints are rechecked.

Step 2b: After randomization of cross-section dimensions of the beams, cross-sectional dimensions of columns are randomized. Until the constraint;

$$b_w \leq b + \frac{3}{2}d \quad (3)$$

is satisfied, randomizations of cross-section of columns continue. In Eq. (3), b is the breadth of the supporting column.

Step 2c: After the dimensions of the frame system is randomly defined, static and dynamic analyses are done in order to find the design loads before the optimum design of reinforcements. Static and dynamic responses are calculated by using the stiffness method and time history analyses. Mass and stiffness matrices of the system are constructed and the structure is not idealized as a shear building. Thus, the dynamic moments resulting from the angular acceleration of structural members are taken into account. In Eqs (4) and (5), mass (M_e) and stiffness (K_e) matrices of a structural member are respectively given. For element matrices; g, E, *l*, I, A and n are gravity, elasticity modulus, length, moment of inertia, cross-sectional area of the element and live load participation factor, respectively.

$$M_e = \frac{\gamma_c Al + (D+nL)(l-a)}{420g}\begin{bmatrix} 140 & 0 & 0 & 70 & 0 & 0 \\ 0 & 156 & 22l & 0 & 54 & -13l \\ 0 & 22l & 4l^2 & 0 & 13l & -3l^2 \\ 70 & 0 & 0 & 140 & 0 & 0 \\ 0 & 54 & 13l & 0 & 156 & -22l \\ 0 & -13l & -3l^2 & 0 & -22l & 4l^2 \end{bmatrix} \quad (4)$$

$$K_e = \begin{bmatrix} \frac{EA}{l} & 0 & 0 & -\frac{EA}{l} & 0 & 0 \\ 0 & \frac{12EI}{l^3} & \frac{6EI}{l^2} & 0 & -\frac{12EI}{l^3} & \frac{6EI}{l^2} \\ 0 & \frac{6EI}{l^2} & \frac{4EI}{l} & 0 & -\frac{6EI}{l^2} & \frac{2EI}{l} \\ -\frac{EA}{l} & 0 & 0 & \frac{EA}{l} & 0 & 0 \\ 0 & -\frac{12EI}{l^3} & -\frac{6EI}{l^2} & 0 & \frac{12EI}{l^3} & -\frac{6EI}{l^2} \\ 0 & \frac{6EI}{l^2} & \frac{2EI}{l} & 0 & -\frac{6EI}{l^2} & \frac{4EI}{l} \end{bmatrix} . \quad (5)$$

The mass (M) and stiffness (K) matrices of structure are obtained by assembling the element matrices and considering global coordinate system. The equation of structures subjected to ground acceleration is written as

$$M\ddot{x}(t) + C\dot{x}(t) + Kx(t) = -M\{1\}\ddot{x}_g(t) \;, \quad (6)$$

where x(t) is deflection vector including displacements and rotations, C is damping matrix calculated by assuming 5% (the value proposed for RC structure) inherent damping for all modes, [1] is a vector of ones with elements as much as degrees of freedom of structure and $\ddot{x}_g(t)$ is ground acceleration. First and second derivatives of x(t)

with respect to time are shown with $\dot{x}(t)$ and $\ddot{x}(t)$, respectively. A Matlab code employing Simulink is developed in order to carry out the time history analyses for the solution of Eq. (6). In the generated code, the coupled equations of motions are separated into vibration modes.
Time history analyses are done for three different earthquake records. The number of earthquakes can be modified according to the situations of numerical applications. After solutions of deflections are obtained, internal forces are calculated by multiplying the stiffness matrix with deflections. Then, dynamic responses are divided to the elastic response parameter (R). The following load combinations (Eqs. (7)-(9)) are calculated for all combinations of live load distributions. From time history analyses, the most unfavorable internal forces are determined by checking seismic analyses for all time lag.

$$U = 1.4D + 1.7L\,. \tag{7}$$
$$U = 0.75(1.4D + 1.7L) \pm E\,. \tag{8}$$
$$U = 0.9D \pm E\,. \tag{9}$$

In these equations, U and E represent total load and earthquake load. The most critical responses are stored for the reinforcement design.
Step 2d: After the internal forces are known, maximum axial force (N_{max}) and maximum shear force (V_{nmax}) are checked for columns. These controls are important for preventing brittle fracture of the elements. V_{nmax} and N_{max} are respectively given in Eqs. (10) and (11).

$$V_{nmax} = \min\begin{Bmatrix} 5.5A \\ 0.2 f_c^{'} A \end{Bmatrix}. \tag{10}$$

$$N_{max} = 0.5 f_c^{'} A\,. \tag{11}$$

For preventing brittle fracture of beams, the maximum axial force capacity is controlled with

$$N_{max} = 0.1 f_c^{'} A\,. \tag{12}$$

If the ductile behaviour conditions are not satisfied, cross-sectional dimensions are randomized again and previous calculations are repeated. Thus, the reinforced concrete design of the system is not checked for the violating constraints of ductile design.
Step2e: After suitable cross-sections are found, longitudinal reinforcements for tensile sections of beams are randomized and the depth of the beam is recalculated. Also, longitudinal reinforcements are randomized in compressive section if the maximum reinforcement ratio (ρ_{max}) given in Eq. (13) is exceeded. Also, ρ_{max} must be less than 0.025.

$$\rho_{max} = (0.75)(0.85)\beta_1 \frac{f_c^{'}}{f_y}\left(\frac{600}{600 + f_y}\right). \tag{13}$$

Positioning of the reinforcement bars is also checked in the optimization process. The clear distance between reinforcements; a_ϕ must satisfy the following conditions;

$$a_\phi > \begin{cases} \phi_{avarage} \\ 25\,mm \\ \frac{4}{3}D_{max} \end{cases}, \tag{14}$$

where D_{max} is the maximum diameter of aggregates and $\phi_{avarage}$ is the average diameter of longitudinal reinforcements. Until these criteria are satisfied, the reinforcements are randomized. The code has ability to place reinforcements in two lines if the clear distance requirement is not provided. In that solution, the value of depth of beam (d) is updated and the required longitudinal reinforcement area is recalculated.
Random reinforcements may not be optimum ones for the randomly chosen cross-section. For that reason, a modification is needed for the design and additional random search iterations are done with a criterion. Randomly chosen reinforcement areas ($A_{s-random}$) must be less than a value which is close to the required reinforcement area ($A_{s-needed}$). This criterion is shown in Eq. (15). By conducting this additional random iteration stage, static and dynamic time history analyses do not need to be repeated. Thus, the feasibility of the consideration of dynamic time history analyses of RC frames is provided.

$$A_{s-random} < (1 + r)A_{s-needed}\,. \tag{15}$$

In Eq. (15), *r* is a user defined value. If the criterion given as Eq. (15) is not provided after 500 iterations, this value is increased by 0.01 after several iterations of random search stage in order to prevent to entrap to a range in which a physical solution cannot be found. For example, the exact area of the reinforcement may not be provided with reinforcement bars with constant diameter sizes. If the total number of iterations of random search exceed 20000, the cost of the beam is taken as a penalized cost (10^6 \$) in order to prevent to entrapping of the optimization. In that case, randomly chosen cross-sectional dimensions are not suitable for the critical internal forces. The numerical values used in this part of the optimization can be changed according to user. In the calculation of the required reinforcement area, minimum longitudinal reinforcement areas ($A_{s,\ min}$) given in Eqs. (16) and (17) are considered.

$$A_{s,\min} \geq \frac{\sqrt{f_c^{'}}}{4 f_y} b_w d \ . \tag{16}$$

$$A_{s,\min} \geq \frac{1.4}{f_y} b_w d \ . \tag{17}$$

According to ACI 318, the conditions given in Eqs. (16) and (17) must be provided in both tensile and compressive sections of the beam at supports and spans. Also, the minimum reinforcement area in compressive section at supports is the half of the area of reinforcement in tensile sections. This requirement must be considered because internal forces may change their direction due to seismic loadings.

After the design of longitudinal reinforcements of the beam, the stirrups are designed. Nominal shear strength of concrete (V_c) and nominal shear strength of reinforcements (V_s) are given in Eqs. (18) and (19).

$$V_c = \frac{\sqrt{f_c^{'}}}{6} b_w d \ . \tag{18}$$

$$V_s = \frac{A_v f_y d}{s} . \tag{19}$$

For ductile response, V_s must not exceed

$$V_s = 0.66\sqrt{f_c^{'}} b_w d \tag{20}$$

Also, minimum shear reinforcement area ($(A_v)_{min}$) is defined as

$$\left(A_v\right)_{\min} = \frac{1}{3}\frac{b_w s}{f_y} . \tag{21}$$

The maximum spacing between shear reinforcements (s_{max}) must be less than

$$s_{\max} \leq \frac{d}{2} \tag{22}$$

but

$$s_{\max} \leq \frac{d}{4} \qquad if \ V_s \geq 0.33\sqrt{f_c^{'}} b_w d \ . \tag{23}$$

For all diameters of reinforcement bars, spacing between bars are calculated. All values of spacing are rounded to values which are multiples of 10 mm. After spacing between stirrups are rounded by considering minimum requirements, exact values of A_v/s are calculated and the design with minimum steel area is chosen. The smallest value with the minimum cost is chosen. After the design variables of beams are assigned, the total cost of the beams is calculated.

Step 2f: Then, the design of reinforcements of columns is done. The effect of slenderness is taken into consideration by using the approximate design procedure given in ACI 318. This procedure uses the moment magnified concept. In second order analyses of frame structures, the deflection needs to represent the state just before the ultimate load. For that reason, moment of inertia of members is reduced by 65% and 30% for beam and columns, respectively.

Also, flexural moment is not taken less than

$$M_{\min} = P_u (15 + 0.03h) \tag{24}$$

where 15 and h are in mm.

In the design of reinforcements of the columns, longitudinal reinforcements are randomly defined for both (upper and lower) faces and a symmetric design is done. As done in reinforcement design of beams, bars can be also placed in two lines. Thus, the placement conditions given in Eq. (25) are checked.

$$a_\phi > \begin{cases} 1.5\phi_{\text{avarage}} \\ 40\ mm \\ \frac{4}{3} D_{\max} \end{cases} . \tag{25}$$

Web reinforcements are also randomized if the clear distance between upper and lower reinforcements is more than 150 mm. Randomizing of reinforcements continues until placement of requirements, minimum (ρ_{min}=0. 01) and maximum (ρ_{max}=0. 06) reinforcement ratios are provided. Also, the optimum reinforcement design for the assigned cross-sectional dimensions is searched by using the following procedure. The distance from extreme compressive fiber to neutral axis (c) is iteratively scanned and the c value for the lowest axial force ensuring the required axial force value is stored. For that value, if the flexural moment capacity ($M_{capacity}$) is lower than the design moment or the optimization constraint given in Eq. (26) is not provided, the randomization of reinforcements iteratively continues.

$$M_{capacity} < (1+r)M_{required}\ . \qquad (26)$$

In Eq. (26), *r* is the same value described in design of beam and it is also iteratively increased as described before. A different parameter can be also used for this stage of optimization. A penalized cost is also considered in optimization of columns (10^6 \$) if a suitable solution cannot be found in 20000 iterations. Also, shear reinforcements are designed similarly as done for the beam design. The costs of columns are calculated and finally the total cost of the structure is obtained. The material cost of an element (C_e) is calculated according to Eq. (27).

$$C_e = (A_g - A_{st})\mathrm{l}_e C_c + (A_{st} + \frac{A_v}{s}u_{st})\mathrm{l}_e \gamma_s C_s\,. \qquad (27)$$

In Eq. (27), C_e is material cost of an element, A_g is the area of cross-section, A_{st} is the area of nonprestressed longitudinal reinforcement, A_v is the area of shear reinforcement spacing s, u_{st} is the length of shear reinforcement spacing s, C_c is the material cost of the concrete per m^3, C_s is the material cost of the steel per ton, l_e is the length of element and γ_s is the specific gravity of steel. The total cost of the structure (objective function=OF) is calculated as

$$OF = \sum_{i=1}^{n}\left(C_e\right)_i \quad \text{n:number of elements.} \qquad (28)$$

After the design variables of beams and columns are generated for all harmony vectors, the initial harmony vectors are combined together in harmony memory (HM) matrix. Harmony memory size is the number of harmony vectors placed in HM matrix. Since additional random search iterations are used in the proposed method, the design variables stored in HM matrix are solutions which are near to the best optimum results. Only a few objective functions may be penalized for minor unphysical solutions.

Step 3: After generation of initial HM matrix, a new harmony vector is generated by using the same procedure. In classical HS, existing harmony vectors are used for the source of newly generated vector with possibility described as harmony memory considering rate (HMCR). In the optimum design of RC frames, a small difference of existing design variables may not provide design restrictions. Thus, in this modified harmony search method, the boundary limits of solution range of cross-section dimensions are updated according to best existing harmony vector. By the change of the ranges, the convergence of optimum result is provided. In order to prevent trapping on local optimum, the initial boundary conditions are used for a structural member with 50% possibility. The lower or upper limit of the range (with equal possibility) is updated with the values of the best harmony vector.

Step 4: If the newly generated vector has a smaller objective function (total cost of the structure) than the existing one, it is replaced with it. Iterations continue until the number of generations reaches an iteration number chosen by the user.

3 Numerical Example

The proposed method was applied to a two-span two-story symmetrical RC frame. Three different earthquake records were used in this study. The information about earthquake records is given in Table 1. Table 2 shows the value of design constants and the ranges used in the optimization of numerical example. Discrete variables are used for design variables and practical design is done. The dimensions are assigned to the values with are the multiples of 50 mm. The diameters of reinforcement bars are assigned to the values which are the multiples of 2 mm. Two-span two-story RC frame is seen in Fig. 2.

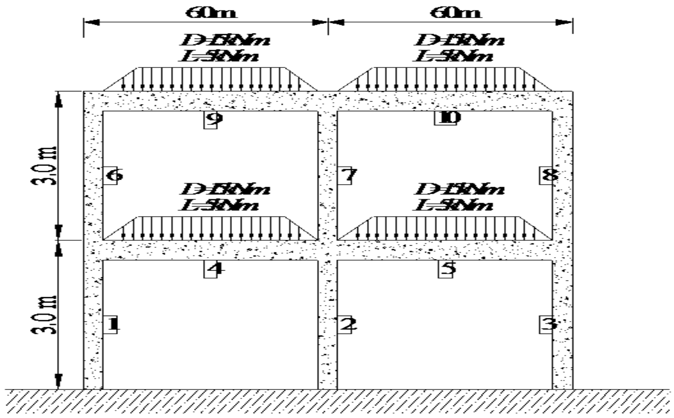

Figure 2 Model of the numerical example

Each span is loaded with 15 kN/m dead load (D) and 5 kN/m live load (L). Live loads were investigated for the unfavorable result in the optimization and the distributed loads were taken as trapezium. For all loadings, the ratio of *a* and *l* (shown in Fig. 1) is taken as ¼.

The optimum results are shown in Table 3 for RC column members. Reinforcements and dimensions of the optimum RC beam are given in Table 4. In the Table 4, LJ and RJ represent left and right joint of the element. The total cost of the RC frame is 304.78 $.

The optimum results of the columns are the same since the dynamic earthquake forces are the critical loading condition for the structure.

TABLE I. EARTHQUAKE RECORDS

Earthquake	Date	Station	Component	PGA(g)
Imperial Valley	1940	117 El Centro	I-ELC180	0.313
Northridge	1994	24514 Sylmar	SYL360	0.843
Loma Prieta	1989	16 LGPC	LGP000	0.563

Note: Earthquake records were taken from PEER NGA DATABASE (http://peer.berkeley.edu/nga/) [37]

TABLE II. DESIGN CONSTANTS AND RANGES OF DESIGN VARIABLES

Definition	Symbol	Unit	Value
Range of web width	bw	mm	250-400
Range of height	h	mm	300-600
Clear cover	cc	mm	30
Range of reinforcement	ϕ	mm	16-30
Range of shear reinforcement	ϕv	mm	8-14
Max. aggregate diameter	Dmax	mm	16
Yield strength of steel	fy	MPa	420
Comp. strength of concrete	f_c'	MPa	25
Elasticity modulus of steel	Es	MPa	200000
Specific gravity of steel	γs	t/m3	7.86
Specific gravity of concrete	γc	t/m3	2.5
elastic response parameter	R	-	8.5
Cost of the concrete per m3	Cc	$	40
Cost of the steel per ton	Cs	$	400

TABLE III. OPTIMUM DESIGN OF COLUMNS

Element Number	bw (mm)	h (mm)	Bars in each face	Shear reinforcement diameter/distance (mm)
1	250	300	2Φ10+ 2Φ12	Φ8/120
2	250	300	2Φ10+ 2Φ12	Φ8/120
3	250	300	2Φ10+ 2Φ12	Φ8/120
6	250	300	2Φ10+ 2Φ12	Φ8/120
7	250	300	2Φ10+ 2Φ12	Φ8/120
8	250	300	2Φ10+ 2Φ12	Φ8/120

TABLE IV. OPTIMUM DESIGN OF BEAMS

Element Number	b_w (mm)	h (mm)	Bars in comp. section	Bars in tensile section	Shear reinforcement diameter/distance (mm)
LJ4			1Φ18+1Φ14 +1Φ16	1Φ20+1Φ30+1Φ14	
4	250	300	2Φ12	1Φ20+1Φ16	Φ8/120
RJ4-LJ5			2Φ16+1Φ12 +1Φ14	1Φ24+1Φ18+1Φ28	
5	250	300	2Φ12	1Φ20+1Φ16	Φ8/120
RJ5			1Φ18+1Φ14 +1Φ16	1Φ20+1Φ30+1Φ14	
LJ9			1Φ18+ 1Φ16	1Φ12+2Φ14+1Φ24	
9	250	300	2Φ12	1Φ14+1Φ26	Φ8/120
RJ9-LJ10			1Φ16+1Φ14 +1Φ20	1Φ22+1Φ24+1Φ16+1Φ18	
10	250	300	2Φ12	1Φ14+1Φ26	Φ8/120
LJ10			1Φ18+ 1Φ16	1Φ12+2Φ14+1Φ24	

The optimum results of symmetric columns and beams must be the same. In that case, the system is optimum if only the design is the same for the symmetric members. For that reason, the results of the numerical example are optimum.

4 Conclusions

Seismic effects are calculated according to the characteristics of the structure. In searching of optimum dimensions of an RC structure, the characteristic of the design has been always changeable. For that reason, seismic loads must be defined according to all random iterations of the optimization process. By using time history analyses for frame structures, a realistic optimum design is done as a novelty of the study. In order to reach that aim, classical metaheuristic algorithms are not feasible in the mean of the computation time. Since a detailed design is done with dynamic analyses, the detailed optimization of the design variables and all members of the frame structure is important. In order to shorten the computation time, random search iterations are needed in some design stages of the proposed method. In the development of the methodology, classical harmony search method was previously used and the unfeasibility of the approach is seen for detailed optimization. The most of the random solutions of members are violating design constraints and the secure solution of the member are not optimum for equal members since the numerical example of the paper is symmetric. Since RC frame structures contain different types of ungrouped structural members such as beams and columns, HS is coupled with additional random search iterations in order to prevent the increase of the number of dynamic analyses. Thus, the optimization process is shortened and probability of finding the optimum design of several members (instead of all members) is neglected.

Since time history analyses in the optimum design of RC structures are not considered in previous studies, the results were compared with the results of first five iterations. A design engineer can also find the results of first five iterations. Thus, the optimum results are compared with these results. The optimum costs are nearly 35% lower than the comparative costs.

The optimization method is feasible and effective to find an optimum RC frame design with minimum cost considering time history analyses, unfavorable loading conditions of live loads and design constraints according to ACI318.

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