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Harmony search algorithm based optimum detailed design of reinforced concrete plane frames subject to ACI 318-05 provisions

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ABSTRACT

This paper presents the application of the harmony search based algorithm to the optimum detailed design of special seismic moment reinforced concrete (RC) frames under earthquake loads based on American Standard specifications. The objective function is selected as the total cost of the frame which includes the cost of concrete, formwork and reinforcing steel for individual members of the frame. The modular sizes of members, standard reinforcement bar diameters, spacing requirements of reinforcing bars, architectural requirements and other practical requirements in addition to relevant provisions are considered to obtain directly constructible designs without any further modifications. For the RC columns, predetermined section database is constructed and arranged in order of resisting capacity. The produced optimum design satisfies the strength, ductility, serviceability and other constraints related to good design and stated in the relevant specifications. The lateral seismic forces are calculated according to ASCE 7-05 and it is updated in each iteration. Number of design examples is considered to demonstrate the efficiency of the optimum design algorithm proposed. It is concluded that the developed optimum design model can be used in design offices for practical designs and this study is the first application of the harmony search method to the optimization of RC frames and also the optimization of special seismic moment RC frames to date.

1. Introduction

In recent years structural optimization has witnessed an emergence of robust and innovative search techniques that strictly avoid gradient-based search to counteract with challenges that traditional optimization algorithms have faced for years. The basic concept behind each of these techniques rests on simulating the paradigm of a biological, chemical, or social systems (such as evolution, immune system, and swarm intelligence or annealing process) that is automated by nature to achieve the task of optimization of its own [1–3]. The design algorithms developed using these meta-heuristic search techniques are particularly suitable for obtaining rapid and accurate solutions to problems in structural engineering discipline [4,5]. This is particularly true in the optimum design of steel structures where the design problem turns out to be a discrete optimization problem when it is formulated according to design codes used in practice.

In many practical engineering design problems, design variables may consist of continuous or discrete variables. In structural optimization problems, the design variables are functions of the cross sections of the members and they are often chosen from a limited set of available values. For example, steel structural members are chosen from standard steel profiles in the market, structural timber members are provided in certain sizes, and reinforced concrete members are usually designed and constructed with discrete dimensional increments. Design optimization of reinforced concrete (RC) structures is more challenging than that of steel structures because of the complexity associated with reinforcement design. Also, only one material is considered for the optimization problems of steel structures and the structural cost is directly proportional to its weight in general. But in the case of the optimum design of concrete structures, three different cost components due to concrete, steel and formwork are to be considered and the overall cost of the structure is affected from any slight variation in the quantity of these components. Therefore, the optimization problem of concrete structures becomes the selection of a combination of appropriate values of design variables to make the total cost component the minimum.

In the literature, there are a number of studies on optimization of RC members and frames. Practical applications of traditional optimization methods are not suitable for optimum design of RC

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frames and these algorithms require additional modifications to fit the discrete nature of structural design variables. Choi and Kwak [6] haves suggested more practical discrete optimization techniques. They used direct search method to select appropriate design sections from some pre-determined discrete member sections for the cost optimization of rectangular beams and columns of RC frames based on the ACI and Korean codes. Balling and Yao used the simulated annealing method which is one of the metaheuristic algorithm to optimize three-dimensional reinforced concrete frames [7]. Discrete variables as well as limits on the number of reinforcing bars and their topological arrangements are considered in their study. Rajeev and Krishnamoorthy [8] applied a simple genetic algorithm to the cost optimization of two-dimensional RC frames. The detailing of reinforcement is considered as a design variable in addition to cross-section dimensions. The allowable combinations of reinforcement bars for columns and beams were tabulated. Camp et al. [9] used genetic algorithms (GAs) by constructing a database for beams and columns which contains the sectional dimensions and the reinforcement data in the practical range to optimize for optimum design of plane frames. Lee and Ahn [10] used to the genetic algorithms to optimize reinforced concrete plane frames subject to gravity loads and lateral loads. In their study, the constructing data sets, which contain a finite number of sectional properties of beams and columns in a practical range removed the difficulties in finding optimum sections from a semi-infinite set of member sizes and reinforcement arrangements. Kwak and Kim [11] studied on optimum design of RC plane frames based on pre-determined section database. In their study, pre-determined section databases of RC columns and beams are constructed and arranged in order of resisting capacity and optimum solutions are obtained using direct search method. They also used genetic algorithms for similar optimization problems [12]. Govindaraj and Ramasamy [13] used genetic algorithms for optimum detailed design for RC frames based on Indian Standard specifications. The dimensions and reinforcement arrangement of column, and the dimensions of beam members alone are considered as a design variables and the detailing of reinforcements in the beam members is carried out as a sub-level optimization problem. The modular sizes of members, available standard reinforcement bar diameters, spacing requirements of reinforcing bars, architectural requirements on member sizes and other practical requirements are arranged in order to obtain for the optimum designs to be directly constructible without any further modifications.

In the studies available in the literature the shear design calculations of concrete members are not considered and the cost of shear reinforcement (ties) is not taken into consideration in the total cost of the frame. Only the simple constraints such as capacity and regulations for flexural reinforcement are included. The detailing of the reinforcement bars is oversimplified and the development length of bars is not considered in the cost calculations. In some of these studies, the lateral loading on the frame is considered; however, the values of the lateral loads are taken as a constant even though the value of lateral loads change with the weight of the structure subject to seismic specifications. Akin and Saka [14] presented optimum design algorithm based on harmony search method for the detailed design of special seismic moment reinforced concrete plane frames considering provisions of ACI 318-05 [15] and ASCE 7-05 [16]. The design algorithm presented in this study considers all required code provisions and obtains the optimum solution by using harmony search algorithm for reinforced concrete frames which is ready for practical design application.

In this study, the algorithm is extended to cover the optimum design of large size reinforced concrete frames. The paper is arranged as follows. In Section 2, the modeling of the detailed optimum design problem is explained; the objective function, the design variables and the constraints derived from design provisions of ACI 318-05 are described. In Section 3, the harmony search algorithm is introduced and the optimum design process of reinforced concrete frames based on the harmony search algorithm is presented. In Section 4, illustrative examples are provided to demonstrate the efficiency and the performance of the algorithm presented in this study. In Section 5, the summary and conclusions are provided.

2. Modeling of optimum detailing design problem of reinforced concrete plane frames

In this study, harmony search method is used to obtain the optimum detailed design for reinforced concrete special seismic moment frames. Reinforced concrete special moment frames are used as part of seismic force-resisting systems in buildings that are designed to resist earthquakes. Beams, columns, and joints in moment frames are proportioned and detailed to resist flexural, axial, and shearing actions during an earthquake. RC frames with special proportioning and detailing requirements are capable of resisting strong earthquake shaking without significant loss of stiffness or strength. These moment-resisting frames are called "Special Moment Frames" because of these additional requirements. A new optimum design algorithm is developed for the RC special seismic moment frames and the variable pool constructed to obtain buildable optimum designs. In the design formulation, the objective function is selected as the cost of the RC structure which includes the cost of concrete, reinforcement and formwork. To satisfy uniformity of the structure and to obtain constructible designs, the beam and column members are separated to design groups. The design variables are categorized into two groups and arranged for the beam and column members. For the columns, the section database which includes the dimensions and the reinforcement detailing of column sections is constructed with the most commonly used sections. The design constraints are implemented according to ACI 318-05. The constraints derived from the code are checked to obtain feasible designs. This study not only considers the flexural design constraints, but also the shear design constraints and the seismic design constraints. The development lengths of the reinforcement steel bars are calculated according to the given regulations in the design specifications. The cost of the shear reinforcement and the impact of the development length on the cost are considered. In the design of frames, the matrix displacement method is used for the structural analysis and the load combinations are taken from the ACI code. The self-weight of RC beams is included in the structural analysis and it is updated in each iteration of the optimization process. The lateral seismic forces are calculated according to ASCE 7-05 and it is updated in each iteration according to the selected design weight.

This paper is unique as it is the first application of the harmony search method to the design optimization of RC frames. Additionally, the detailing of the reinforcement in the concrete members, the consideration of the shear design of members, and the derivation of the constraints are handled in more detail in this study. The lateral seismic forces affecting the RC frame are obtained for the site properties according to ASCE 7-05 even though other existing studies do not consider the lateral seismic forces in their design. Also, this study is the sole study about the optimization of *Special Seismic Moment RC Frames*to date.

2.1. Objective function

In structural optimization problems, the objective function is generally described as the weight or total cost of structure. Usually, the weight of structure is used for the optimum design of steel structures. For the optimum design of reinforced concrete (RC) structures, the cost of structure is more convenient as an objective function, because concrete structures involve different materials and the nature of concrete design is different. In reality, the minimum weight design may not be the minimum cost design for especially concrete structures. Besides the unit costs of different materials involved concrete construction influence the total cost of the concrete structures. For these reasons, the optimization problem of concrete structures should be formulated in terms of the total cost, which includes the costs of concrete, steel and formwork.

In this study, the objective function is selected as the total cost consisting of individual cost components of concrete, steel and formwork. The cost of any component is inclusive of material, fabrication, and labor. The objective function is expressed mathematically,

Minimize

$$f_{cost} = C_c + C_s + C_f \tag{1}$$

where C_c = the cost of concrete, C_s = the cost of reinforcing bars, C_f = the cost of formwork (includes labor and placement).

The costs of components, C_c, C_s, and C_f, are calculated for each member by the following equations. For the cost of concrete, C_c;

$$C_c = V_c \cdot (UC_c) \tag{2}$$

where C_c = the cost of concrete, V_c = the total volume of concrete, (UC_c) = the unit cost of concrete (\$/m^3).

$$V_c = \sum_{i=1}^{N_{col}} b_i d_i L_{column,i} + \sum_{j=1}^{N_{beam}} b_{w,j} h_j L_{clear\ beam,j} \tag{3}$$

where N_{col} = the number of column members, b_i and d_i = the width and depth of ith column member with rectangular cross section, $L_{column,i}$ = the length of ith column member, N_{beam} = the number of beam members, $b_{w,j}$ = the width of jth beam, h_j = the height of jth beam, $L_{clear\ beam,j}$ = the clear span length of jth beam.

For the cost of steel, C_s;

$$C_s = W_s \cdot (UC_s) \tag{4}$$

where C_s = the cost of steel, W_s = the total weight of steel used as reinforcement bar in the concrete frame, (UC_s) = the unit cost of steel (\$/kg).

$$W_s = \sum_{i=1}^{N_{col}} \sum_{j=1}^{N_{bar,i}} A_{s,j} l_{bar,j} + \sum_{i=1}^{N_{col}} \sum_{k=1}^{N_{tie,i}} A_{sh,k} l_{tie,k} + \sum_{m=1}^{N_{beam}} \sum_{l=1}^{N_{bar,m}} A_{s,l} l_{bar,l} + \sum_{m=1}^{N_{beam}} \sum_{n=1}^{N_{tie,m}} A_{st,n} l_{tie,n} \tag{5}$$

where $N_{bar,\ldots}$ = the number of longitudinal reinforcement bars placed in the member, $N_{tie,\ldots}$ = the number of ties used in the member, $A_{s,\ldots}$ = the area of reinforcement bars, $l_{bar,\ldots}$ = the length of reinforcement bars, $A_{st,\ldots}$,$A_{sh,\ldots}$ = the area of shear reinforcement bars (ties) $l_{tie,\ldots}$ = the length of ties.

For the cost of formwork, C_f;

$$C_f = A_f \cdot (UC_f) \tag{6}$$

where C_f = the cost of formwork, A_f = the total formwork area, (UC_f) = the unit cost of formwork (\$/m^2).

$$A_f = \sum_{i=1}^{N_{col}} \{2(b_i + d_i)L_{column,i} - (Area_{beams@joint,i})\} + \sum_{j=1}^{N_{beam}} \{(b_{w,j} + 2h_j)L_{clear\ beam,j}\} \tag{7}$$

where Area$_{beams@joint,i}$ = the cross-section area of beams connected to the ith column at joint.

The detailed evaluation of objective function is defined by Eqs. (2–7). The unit costs are based on market prices and their values changes from time to time and also from country to country. For these reasons, the unit price data cannot be fixed and needs to be updated. In the previous studies in the literature, researchers used different country design specifications for design and different market prices for unit costs depending to their countries. The different unit material costs are used for the design examples, and these values are given with concerned examples.

2.2. Design variables

Design variables are divided two groups as column design variables and beam design variables and to obtain practical designs beams and columns are separated to groups. Total number of design variables is determined according to number of column and beam groups.

2.2.1. Beam design variables

For beam design groups, the cross-section of beams, the area of reinforcement bars along all beams, and the area of reinforcement bars placed on the top and bottom of beam spans and supports are considered as design variables. The cross-sectional dimensions of the beam are considered as the design variable. In addition to cross-section dimension of the beam, the areas of longitudinal bars that are placed continuously at the bottom of all the beams and the tensile reinforcements at the spans of beams, and supports for each beam are also considered as the design variable. These design variables and their numbers in the problem are defined in Table 1. The design variables relevant with reinforcement bars are not defined as the surface area of reinforcing bars. Instead the reinforcement bar layout is defined and these design variables relevant with reinforcement are expressed in terms of the number of reinforcing bars and the diameter of reinforcing bars. By this way, design process can reach directly constructible optimum designs. The details of the design variables are shown in Fig. 1. The total number of the beam design variables, N_{bdv}, for one beam group changes according to number of span (bay) in frame. Total number of beam design variables for one beam group is calculated by Eq. (8);

$$N_{bdv} = 5 + 4n_{bay} \tag{8}$$

The total number of beam design variables when each beam group considered, Ndv_{beam}, is computed by following equation;

$$Ndv_{beam} = n_{beamgroup} \cdot N_{bdv} \tag{9}$$

where N_{bdv} = the total number of the beam design variables for one beam group, n_{bay} = the number of bays in a frame, $n_{beam\ group}$ = the total number of beam group.

For the cross-sectional and reinforcement design variables, the design variable pools are created as shown in Table 2. For the reinforcement design variables, the values are selected such that they give constructable reinforcement areas. In other words, the number and diameter of the bar for a beam can be considered as one design variable (*n*ϕ*d n; number of bars, d; diameter of bars*). For this purpose, the variable pool table is composed of a combination of number and diameter of bars. For the reinforcement design variables, the design variable pool is created by the combination of number and diameter of reinforcement bars as shown in Table 2. It is apparent from the design pool that 61 different combination of number and diameter of a bar can be arranged for each reinforcement design variables. The 6 reinforcement bar combination are used for the reinforcement continues through the top ($X_{i,2}$) and bottom ($X_{i,3}$) of all the beams. In this optimum design problem, material strengths, unit costs of materials, structural geometry, support conditions, loading conditions, and cover details are preassigned as design parameters at the beginning of the optimization

Table 1
Cross-sectional and reinforcement design variables for beam groups.

Design variables		Number
$X_{i,1}$	The width and the height of beam cross-section	1
$X_{i,2}$	The area of the steel reinforcement that continues through the top of all the beams[a]	1
$X_{i,3}$	The area of the steel reinforcement that continues through the bottom of all the beams[a]	1
$X_{i,4} - X_{i,k}$	The area of the top steel reinforcement at spans of beams[a]	n_{bay}
$X_{i,k+1} - X_{i,m}$	The area of the bottom steel reinforcement at spans of beams[a]	n_{bay}
$X_{i,m+1} - X_{i,n}$	The area of the top steel reinforcement at the supports[a]	n_{bay}+1
$X_{i,n+1} - X_{i,Nbdv}$	The area of the bottom steel reinforcement at supports[a]	n_{bay}+1

i = number of beam group (ith beam group), n_{bay}; number of spans ($k = 3 + n_{bay}$, $m = 3 + 2n_{bay}$, $n = 4 + 3n_{bay}$, $N_{bdv} = 5 + 4n_{bay}$).
[a] For the reinforcement design variables, the areas of steel reinforcement bars are defined in terms of the number and diameter of the bars ($n\phi d$ *n; number of bars, d; diameter of bars*) to obtain constructible reinforcement areas.

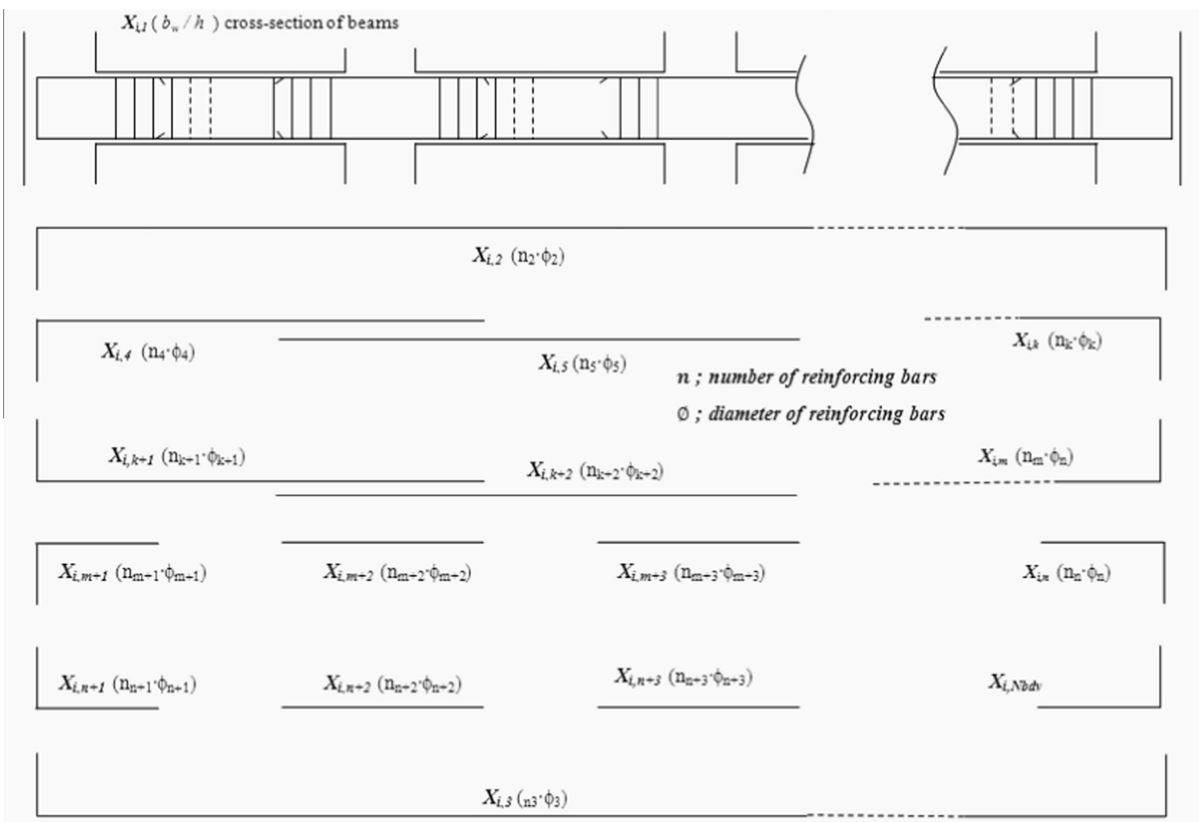

Fig. 1. The reinforcement bar layout and the design variables for ith beam group.

process. However, the value of the dead load which includes the self-weight of beam depending on the cross-sectional dimensions is automatically updated during the design cycles.

2.2.2. Column design variables

The optimum design problem of RC buildings is more complex than the optimal design of steel structures because only one structural material is considered in steel structures and the cost of the steel structures is only assumed to be proportional to its weight. Unlike the design of steel structures, there is infinite set of member size and amount of steel reinforcement used in the design of reinforced concrete (RC) buildings. The importance difference between optimum designs of steel and concrete structures is that more combinatorial characteristics affect the determining the cross-sectional dimensions, and the layout and arrangements of reinforcing bars in RC building design. In addition to the discrete and combinatorial nature of the RC sections, the restrictions and reinforcement detailing specified in the design specifications make the optimum design of RC buildings even more complicated.

In the literature, many researchers suggested and used a lot of practical discrete optimization techniques developed for discrete optimum design of RC frames by the construction of a concrete section database [6–10]. In these studies, discrete optimum sections are directly searched based on the relationship between the section identification numbers and properties (dimensions and resistant capacities) of sections. The sections widely used in practical design are selected to construct database.

In this study, the concrete section database is constructed for the selection of section of column members. Practically, the dimensions of column sections in a RC frames are usually increased by 50 mm a step, and the diameters of reinforcing bars in column members change between 14 and 30 mm most frequently. The design variables for the construction of column section database are selected as the dimensions of columns in x and y directions, the diameter of reinforcement bars at the cross-section of column and numbers of reinforcement bars in both sides of the column as shown in Fig. 2. The lower–upper bounds and increments of these variables are given in Table 3. Assigning these values to variables

Table 2
Variable poll for beam design variables.

#	X_1 (mm)	#	X_2-X_3	#	X_4-X_{Nbdv}
1	250/400	1	2ϕ12	1	*NNR*
2	250/450	2	2ϕ14	2	1ϕ12
3	250/500	3	3ϕ12	3	1ϕ14
4	250/550	4	2ϕ16	4	1ϕ16
5	250/600	5	3ϕ14	5	1ϕ18
6	300/400	6	3ϕ16	6	1ϕ20
7	300/450			7	1ϕ22
8	300/500			8	1ϕ24
9	300/550			9	1ϕ26
10	300/600			10	1ϕ28
11	300/650			11	1ϕ30
12	300/700			12	2ϕ12
13	350/500			13	2ϕ14
14	350/550			14	2ϕ16
15	350/600			15	2ϕ18
16	350/650			16	2ϕ20
17	350/700			17	2ϕ22
18	400/600			.	.
19	400/650			.	.
20	400/700			45	5ϕ18
21	400/750			46	5ϕ20
22	400/800			47	5ϕ22
23	450/700			48	5ϕ24
24	450/750			49	5ϕ26
25	450/800				
26	450/850				
27	450/900			55	6ϕ18
28	500/700			56	6ϕ20
29	500/750			57	6ϕ22
30	500/800			58	6ϕ24
31	500/850			59	6ϕ26
32	500/900			60	6ϕ28
				61	6ϕ30

[a] *NNR*; no need reinforcement.

all possible section combinations are composed, and the combinations satisfy the constraints between 12 and 20 (given in Section 2.3.1) are taken the section pool. However, the number of possible combinations is 37,440 and obtained feasible sections number is obtained as 6199. This quantity of feasible column sections is too large to obtain the optimum results using optimization methods. Additionally, some of sections have similar section properties and flexural moment strength. For these reasons, obtained feasible column sections are evaluated to their section properties and flexural strength for balanced case. Finally, the feasible solution number in the column section database is decreased to 219 which is a reasonable number for optimum design process. The selected feasible column sections are given in Table 4. The compressive strength of concrete and the yield strength of steel are taken as 30 MPa and 400 MPa, respectively, and the cover of the concrete section is taken as 50 mm to evaluate the strength of sections and to select the sections in the database. In the optimum design process, the interaction diagrams for the assigned sections are computed for the given material strength properties and the detailing of the section.

2.3. Constraints

Constraints to be considered in the optimum design problem are strength, serviceability, ductility and other side constraints. Constraints can be imposed separately for column groups, beam groups and connection regions. These constraints are taken from ACI 318-05.

2.3.1. Constraints for column groups

Two types of design constraints are considered for the column members of the RC frame. The first type includes those constraints on the axial load and moment resistance capacities of the section, clear spacing limits between reinforcing bars, and the minimum and the maximum percentage of steel allowed. The second type consists of those constraints defining architectural requirements and good design and detailing practices. These include the requirement of the minimum and the maximum dimensions of column, the maximum aspect ratio of the section, maximum

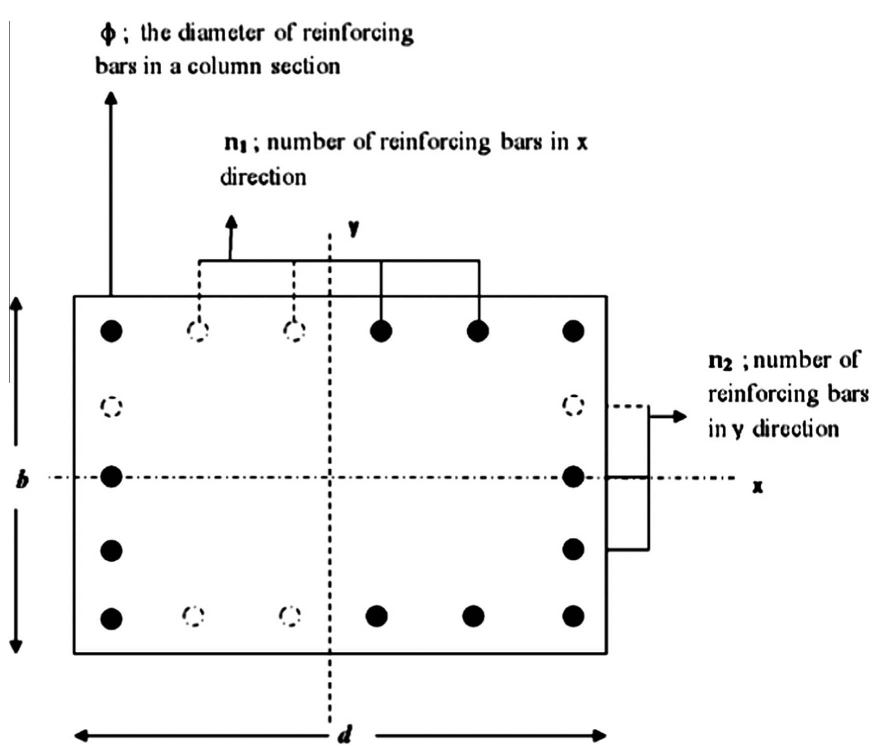

Fig. 2. The design variables for construction of column section database.

Table 3
Design variable bounds for RC column design examples.

Design variables		Lower bound	Upper bound	Increment (step size)	Number of possible values in the range
X_{1c}	b (mm)	300	500	50 mm	5
X_{2c}	d (mm)	400	1000	50 mm	13
X_{3c}	ϕ (mm)	{14,16,18,20,22,24,26,28,30}			9
X_{4c} and X_{5c}	n_1 and n_2	0	7	1	8

Table 4
Design variables database for column sections.

#	d	b	ϕ	n_1	n_2	$P_{n,\max}$ (kN)	P_b (kN)	M_b (kN m)
1	400	300	14	1	1	2288.78	851	130.97
2	400	300	14	1	2	2363.72	848.46	142.13
3	400	350	14	2	1	2695.22	991.85	149.3
4	400	350	14	1	2	2695.22	991	158.38
5	400	350	22	1	1	3060.77	972.18	193.67
6	400	400	16	2	1	3141.44	1128.26	175.97
7	400	400	18	1	1	3147.55	1125.48	184.34
.	.	.		.		.	.	.
.	.	.		.		.	.	.
31	550	300	20	4	1	3805.51	1248.66	316.54
32	550	300	20	2	2	3652.57	1222.19	330.19
33	550	350	14	5	2	3865.19	1456.02	305.28
34	550	350	14	3	3	3790.25	1441.67	311.33
35	550	350	20	2	1	3955.43	1431.02	326.84
.	.	.		.		.	.	.
.	.	.		.		.	.	.
59	600	400	22	3	2	5273.47	1818.46	504.35
60	600	400	18	4	4	5216.88	1816.61	508.45
61	600	450	16	3	2	5160.46	2043.57	444.72
.	.	.		.		.	.	.
.	.	.		.		.	.	.
79	650	500	20	5	3	6916.36	2505.33	696.33
80	650	500	20	3	6	7069.31	2459.97	795.58
81	650	500	24	3	5	7589.33	2459.47	889.61
82	700	300	14	5	1	4080.31	1641.18	399.37
83	700	300	20	4	2	4704.34	1640.9	527.01
84	700	300	20	5	2	4857.29	1662.59	543.7
.	.	.		.		.	.	.
.	.	.		.		.	.	.
111	750	300	16	4	1	4414.58	1762.82	472.77
112	750	300	14	5	2	4403.88	1766.52	477.93
113	750	300	16	5	1	4512.47	1778	484.58
114	750	300	18	5	1	4720.48	1788.23	520.22
.	.	.		.		.	.	.
.	.	.		.		.	.	.
141	800	350	20	4	2	5864.59	2208.09	725.95
142	800	350	20	5	2	6017.54	2232.38	745.8
143	800	350	18	7	3	6127.66	2246.81	767.84
144	800	350	20	7	2	6323.43	2268.88	783.91
.	.	.		.		.	.	.
.	.	.		.		.	.	.
158	800	500	24	5	5	9272.95	3154.95	1299.31
159	850	350	14	7	1	5680.51	2371.26	664.1
160	850	350	16	7	1	5909.93	2386.99	707
161	850	350	14	7	3	5830.4	2366.19	721.87
162	850	350	16	6	2	5909.93	2369.95	731.11
.	.	.		.		.	.	.
.	.	.		.		.	.	.
187	900	450	20	5	3	8242.36	3226.06	1141.62
188	900	500	20	5	5	9294.13	3564.72	1350.11
189	950	400	20	5	1	7522.09	3057.72	1027.23
.	.	.		.		.	.	.
.	.	.		.		.	.	.
216	1000	500	20	5	3	9816.98	3998.45	1473.01
217	1000	500	18	5	5	9774.16	3977.65	1504.87
218	1000	500	22	6	4	10508.3	4025.99	1677.74
219	1000	500	28	5	2	10985.5	4071.49	1749.45

number of reinforcing bars and other reinforcement requirements. The constraints are explained and expressed in a normalized form as given below.

Two types of design constraints are considered for the column members of the RC frame. The first type includes those constraints on the axial load and moment resistance capacities of the section, clear spacing limits between reinforcing bars, and the minimum and the maximum percentage of steel allowed. The second type consists of those constraints defining architectural requirements and good design and detailing practices. These include the requirement of the minimum and the maximum dimensions of column, the maximum aspect ratio of the section, maximum number of reinforcing bars and other reinforcement requirements. The constraints are explained and expressed in a normalized form as given below.

The maximum axial load capacity of columns, $P_{n,\max}$ should be greater than the factored axial design load acting on the column section, P_d;

$$g_1(x) = \frac{P_{d(i,j)}}{P_{n,\max}(i,j)} - 1 \leqslant 0 \quad i = 1, N_{col} \quad j = 1,6 \tag{10}$$

where i = number of the column (ith column), N_{col} = total number of columns, j = load combination type.

For a column section with uni-axial bending, the moment carrying capacity of column section, M_n, obtained for each factored axial design load, P_d, should be greater than the applied factored design moment, M_d;

$$g_2(x) = \frac{M_{d(i,j)}}{M_{n@P_{u(i,j)}}} - 1 \leqslant 0 \qquad i = 1, N_{col} \quad j = 1,6 \tag{11}$$

where i = number of the column (ith column), N_{col} = total number of columns, j = load combination type.

The percentage of longitudinal reinforcement steel, ρ, in a column section should be between minimum and maximum limits permitted by design specification ($\rho_{\min}$ = 0.01 and $\rho_{\max}$ = 0.06);

$$g_3(x) = \frac{\rho_{\min}}{\rho_i} - 1 \leqslant 0 \tag{12}$$

and

$$g_4(x) = \frac{\rho_i}{\rho_{\max}} - 1 \leqslant 0 \qquad i = 1, N_{col} \tag{13}$$

where i = number of the column (ith column), N_{col} = total number of columns.

The width b and the height d of a column section should not be less than the minimum dimensions limit value given for columns (*min. dimension*, $cd_{\min}$ = 300 mm);

$$g_5(x) = \frac{cd_{\min}}{b_i} - 1 \leqslant 0 \tag{14}$$

and

$$g_6(x) = \frac{cd_{\min}}{d_i} - 1 \leqslant 0 \qquad i = 1, N_{col} \tag{15}$$

where i = number of the column (ith column), N_{col} = total number of columns.

The ratio of shorter dimension of column section to longer one should be greater than permitted limit ($cdr_{\min}$ = 0.40);

$$g_7(x) = \frac{cdr_{\min}}{(b_i/d_i)} - 1 \leqslant 0 \qquad i = 1, N_{col} \tag{16}$$

where i = number of the column (ith column), N_{col} = total number of columns.

The minimum diameter of longitudinal reinforcing bars, ϕ, in a column section should be greater than minimum bar diameter, $\phi_{\min}$, specified by design code;

$$g_8(x) = \frac{\phi_{\min}}{\phi_i} - 1 \leqslant 0 \qquad i = 1, N_{col} \tag{17}$$

where i = number of the column (ith column), N_{col} = total number of columns.

The total number of longitudinal reinforcing bars, nrb, in a column section should be smaller than specified maximum number of reinforcing bars, $nrb_{\max}$, for detailing practice ($nrb_{\max}$ = 24);

$$g_9(x) = \frac{nrb_i}{nrb_{\max}} - 1 \leqslant 0 \qquad i = 1, N_{col} \tag{18}$$

where i = number of the column (ith column), N_{col} = total number of columns.

The minimum and maximum clear spacings between longitudinal bars, a, in a column section should be between minimum and maximum limits, $a_{\min}$ and $a_{\max}$, specified for detailing practice ($a_{\min}$ = 50 mm and $a_{\max}$ = 150 mm);

$$g_{10}(x) = \frac{a_{\min}}{a_i} - 1 \leqslant 0 \tag{19}$$

and

$$g_{11}(x) = \frac{a_i}{a_{\max}} - 1 \leqslant 0 \qquad i = 1, N_{col} \tag{20}$$

where i = number of the column (ith column), N_{col} = total number of columns.

The shear force capacity of column section, ϕV_n, should be greater than applied factored design shear force, V_d;

$$g_{12}(x) = \frac{V_{d(i,j)}}{\phi V_{n(i,j)}} - 1 \leqslant 0 \qquad i = 1, N_{col} \quad j = 1, 6 \tag{21}$$

where i = number of the column (ith column), N_{col} = total number of columns, j = load combination type.

Also, the shear force capacity of column section, ϕV_n, should be greater than the minimum capacity shear forces, $\min\{V_e^c, V_e^b\}$, based on probable maximum flexural strength of column, V_e^c , and based on probable maximum flexural strengths at the ends of beams framed into the top joint of column, V_e^b;

$$g_{13}(x) = \frac{\min\left\{ V_{e(k)}^c \quad V_{e(k)}^b \right\}}{\phi V_{n(k)}} - 1 \leqslant 0 \quad k = 1, N_{column\ group} \tag{22}$$

where k = number of the column group (kth column group), N_{col} = total number of column groups.

The factored design shear force acting on column section,V_d, should be less than allowed maximum shear force capacity, $\phi V_{\max}$;

$$g_{14}(x) = \frac{V_{d(i,j)}}{\phi V_{\max\ (i,j)}} - 1 \leqslant 0 \quad i = 1, N_{col} \quad j = 1, 6 \tag{23}$$

where i = number of the column (ith column), N_{col} = total number of columns, j = load combination type.

The area of shear reinforcement (ties), A_{sh}, should be greater than limitations on the minimum area of shear reinforcement, $A_{sh,\min}$;

$$g_{15}(x) = \frac{A_{sh,\min}}{A_{sh(i,j)}} - 1 \leqslant 0 \qquad k = 1, N_{column\ \in group} \quad j = 1, 3 \tag{24}$$

$$A_{sh,\min} \rightarrow \max \begin{cases} 0.062\sqrt{f_c}bd\frac{s}{f_y} \geqslant 0.35d\frac{s}{f_y} \\ 0.3sb_c\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f_c'}{f_y} \\ 0.09sb_c\frac{f_c}{f_y} \end{cases} \tag{25}$$

where k = number of the column group (kth column group), $N_{column\ group}$ = total number of column groups, j = three (top, middle and bottom parts) shear design region of column, f_c = the compressive strength of concrete, f_y = the yielding strength of reinforcing steel, b and$d$ = the width and height of column section, s = the spacing between stirrups (ties), b_c = the cross-sectional dimension of column core measured center-to-center of outer legs of the transverse reinforcement comprising area A_{sh}, A_g = the gross area of section, A_{ch} = the cross-sectional area of member measured out-to-out of transverse reinforcement.

The spacing between stirrups, s, in the column should be greater than minimum spacing, $s_{\min}$ ($s_{\min}$ = 50 mm) for constructional requirements;

$$g_{16}(x) = \frac{s_{\min}}{s_{(i,j)}} - 1 \leqslant 0 \qquad k = 1, N_{column\ group} \quad j = 1, 3 \tag{26}$$

where k = number of the column group (kth column group), $N_{column\ group}$ = total number of column groups, j = three (top, middle and bottom parts) shear design region of the column.

At the top and bottom ends of column members, the spacing between stirrups, s, should be less than maximum spacing of shear reinforcement for end regions of column, $s_{\max,end}$;

$$g_{17}(x) = \frac{s_{(i,j)}}{s_{\max,end}} - 1 \leqslant 0 \qquad k = 1, N_{column\ group} \quad j = 1, 2 \tag{27}$$

$$s_{\max,end} \rightarrow \min \begin{cases} \frac{d}{4} \\ \frac{b}{4} \\ 6\phi_b \end{cases} \tag{28}$$

where k = number of the column group (kth column group), $N_{column\ group}$ = total number of column groups, j = two (top and middle parts) shear design region of the column, b and$d$ = the width and height of column section, s = the spacing between stirrups (ties), ϕ_b = the diameter of longitudinal reinforcing bars.

For the middle parts column members, the spacing between stirrups, s, should be less than maximum spacing of shear reinforcement, $s_{\max,middle}$;

$$g_{18}(x) = \frac{s_{(i,middle)}}{s_{\max,middle}} - 1 \leqslant 0 \qquad k = 1, N_{column\ group} \tag{29}$$

$$s_{\max,middle} \rightarrow \min \begin{cases} 6\phi_b \\ 15\ \text{cm} \end{cases} \tag{30}$$

where k = number of the column group (kth column group), $N_{column\ group}$ = total number of column groups, ϕ_b = the diameter of longitudinal reinforcing bars.

The length of top and bottom shear regions, l_o, should be greater than allowable design length, $l_{o,\min}$;

$$g_{19}(x) = \frac{l_{0,\min}}{l_{0(i,j)}} - 1 \leqslant 0 \qquad k = 1, N_{column\ group} \quad j = 1, 2 \tag{31}$$

$$l_{0,\min} \rightarrow \max \begin{cases} \textit{larger of } b \textit{ and } d \\ 1/6 \textit{ clear span of column} \\ 45\ \text{cm} \end{cases} \tag{32}$$

where k = number of the column group (kth column group), $N_{column\ group}$ = total number of column groups, j = two (top and middle parts) shear design region of column, b and$d$ = the width and height of column section.

2.3.2. Constraints for beam groups

Constraints to be imposed for each beam group are based on strength, serviceability, ductility and other side constraints. The normalized forms of all constraints considered in optimum design problem are given below.

For three critical sections (left end, middle part and right end) of each beam, the negative (with top steel in tension) and positive (with bottom steel in tension) moment carrying capacities of section for M_n, should be greater than the applied factored design moments M_d;

$$g_{20}(x) = \frac{M_{d(i,j,k,l)}}{M_{n(i,j,k,l)}} - 1 \leqslant 0 \quad i = 1, N_{beam} \quad j = 1, \ldots, 6k = 1, 3l = 1, 2 \tag{33}$$

where i = number of the beam (ith beam), N_{beam} = total number of beams, j = load combination type, k = three critical sections for flexural design of beam, l = the negative moment and positive moment situations (top steel in tension or bottom steel in tension).

The tension area of longitudinal reinforcement steel bars in tension A_s, for three critical sections in a beam should satisfy the minimum and the maximum requirements permitted by design specification;

$$g_{21}(x) = \frac{A_{s,\min\ (i,k,l)}}{A_{s(i,k,l)}} - 1 \leqslant 0 \qquad i = 1, N_{beam} \quad k = 1, 3 \quad l = 1, 2 \tag{34}$$

and

$$g_{22}(x) = \frac{A_{s(i,k,l)}}{A_{s,\max\ (i,k,l)}} - 1 \leqslant 0 \qquad i = 1, N_{beam} \quad k = 1, 3 \quad l = 1, 2 \tag{35}$$

$$A_{s,\min} \rightarrow \max \begin{cases} 0.25\frac{f'_c}{f_y} b_w h \\ 1.4\frac{b_w h}{f_y} \end{cases}$$
$$A_{s,\max} = 0.025 b_w h \quad (\textit{for the total area of top and bottom flexural bars}) \tag{36}$$

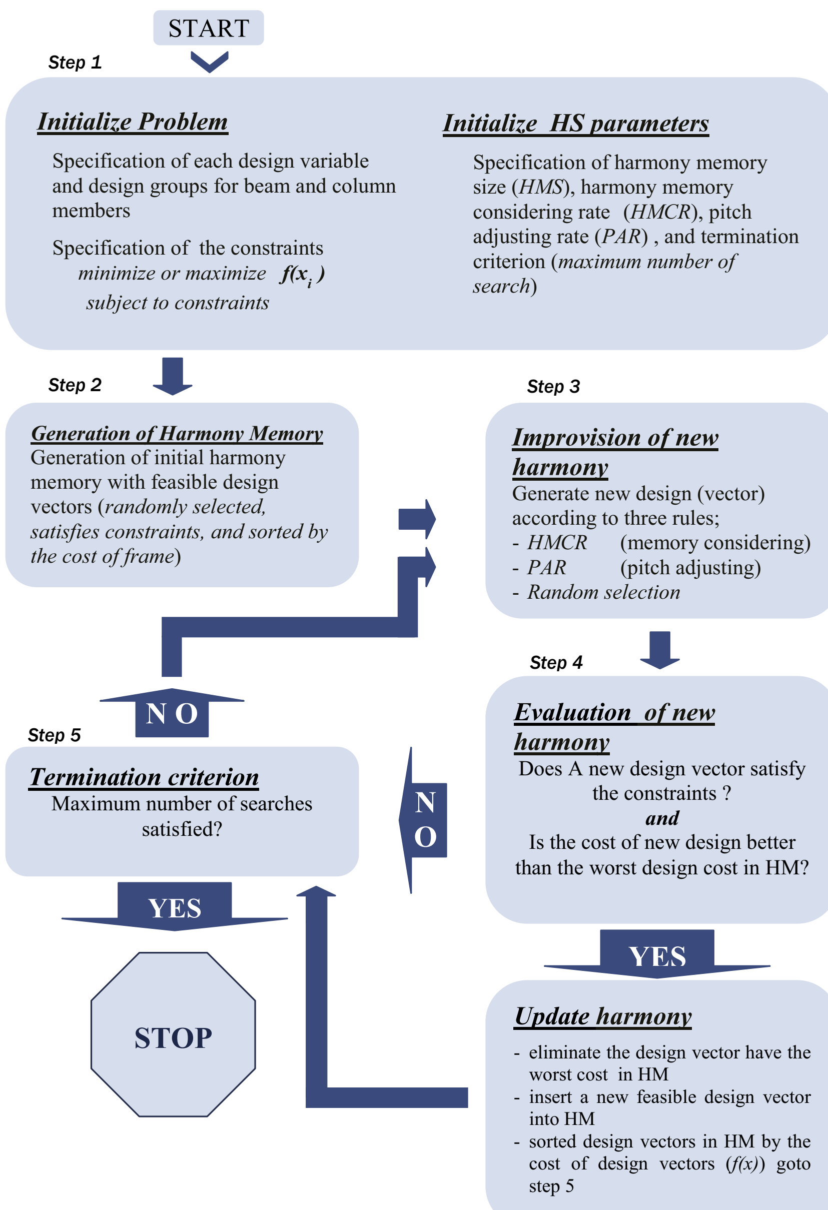

Fig. 3. Optimum design procedure of RC frames with Harmony search algorithm.

where i = number of the beam (ith beam), N_{beam} = total number of beams, k = three critical sections for flexural design of beam, l = the negative moment and positive moment situations (checking for top and bottom reinforcement steel area), b_w and$h$ = the width and height of beam section, f_c = the compressive strength of concrete,f_y = the yielding strength of reinforcing steel.

At any end (support) of the beams, the positive moment capacity M_n^-, (i.e., associated with the bottom steel) should be greater than 1/2 of the beam negative moment capacity M_n^+, (i.e., associated with the top steel) at that end;

$$g_{23}(x) = \frac{\frac{1}{2}M^-_{n,(i,k)}}{M^+_{n,(i,k)}} - 1 \leqslant 0 \qquad i = 1, N_{beam} \; k = 1,2 \tag{37}$$

where i = number of the beam (i.th beam), N_{beam} = total number of beams, k = the left and right ends of beam.

The positive flexural moment strength,$M^+_{n,middle}$, at span of beams should be greater than a quarter of negative and positive flexural moment strengths at the ends of beams, $M^-_{n,}$ and M^+_n ;

$$g_{24}(x) = \frac{\frac{1}{4}M^-_{n,(i,k)}}{M^+_{n,(i,middle)}} - 1 \leqslant 0 \tag{38}$$

and

$$g_{25}(x) = \frac{\frac{1}{4}M^+_{n,(i,k)}}{M^+_{n,(i,middle)}} - 1 \leqslant 0 \qquad i = 1, N_{beam} \; k = 1,2 \tag{39}$$

where i = number of the beam (i.th beam), N_{beam} = total number of beams, k = the left and right ends of beam.

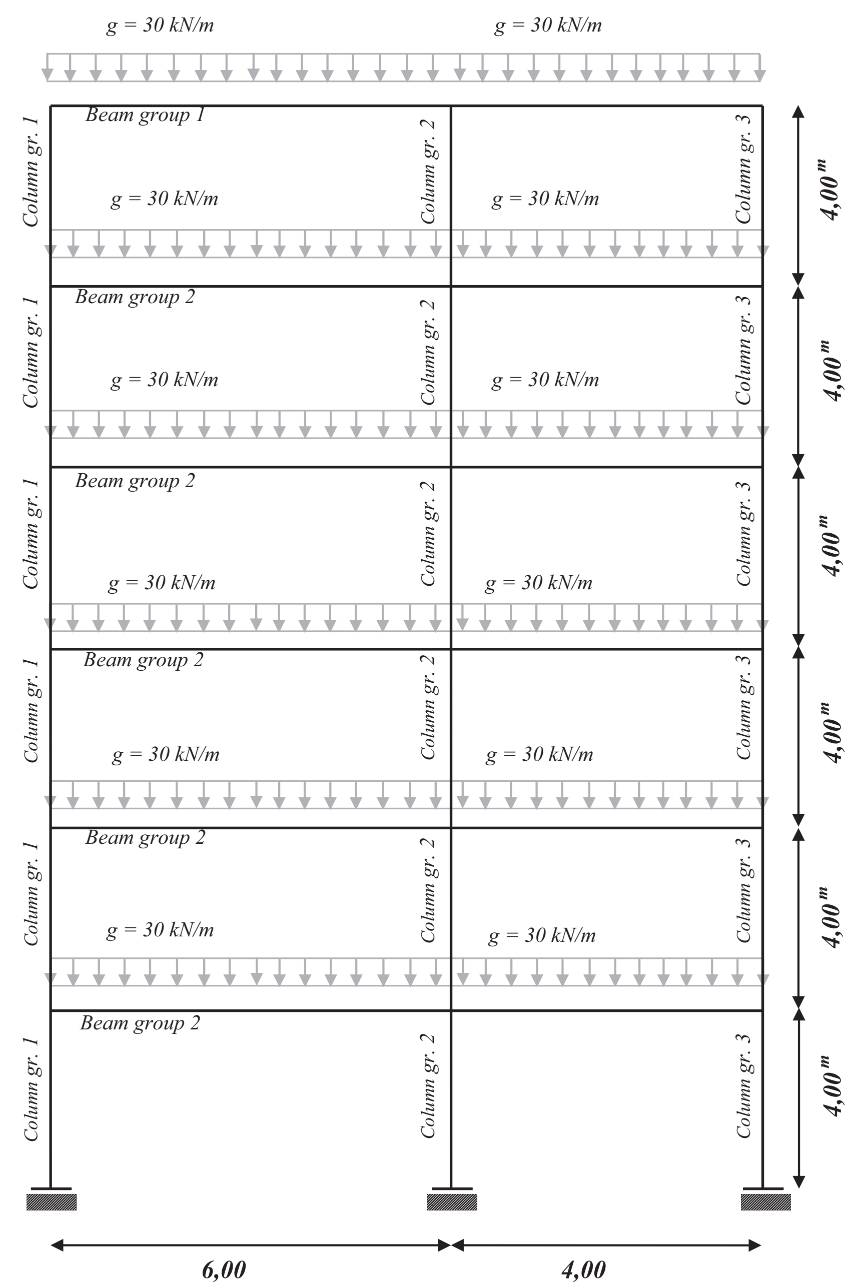

Fig. 4. Two-bay six-storey RC plane frame.

The shear force capacity of three regions (left and right ends, and span) of beam, ϕV_n, should be greater than applied factored design shear force, V_d;

$$g_{26}(x) = \frac{V_{d(i,j,k)}}{\phi V_{n(i,j,k)}} - 1 \leqslant 0 \qquad i = 1, N_{beam} \quad j = 1,6 \quad k = 1,3 \qquad (40)$$

where i = number of the beam (i.th beam), N_{beam} = total number of beams, j = load combination type, k = three critical region (left and right ends, and span) for shear design of beam.

Also, the shear force capacity of design regions in a beam, ϕV_n, should be greater than the probable shear forces based on probable maximum flexural strengths with the factored gravity loads at beam ends of beam, max{V_{e1},V_{e2}};

$$g_{27}(x) = \frac{\max\{V_{e1(i)} \quad V_{e1(i)}\}}{\phi V_{n(i,k)}} - 1 \leqslant 0 \qquad i = 1, N_{beam} \quad k = 1,3 \qquad (41)$$

where i = number of the beam (i.th beam), N_{beam} = total number of beams, k = three critical region (left and right ends, and span) for shear design of beam.

The factored design shear forces at the middle and ends of beam,V_d, should be less than allowed maximum shear force capacity, $\phi V_{\max}$;

$$g_{28}(x) = \frac{V_{d(i,k)}}{\phi V_{\max\ (i,k)}} - 1 \leqslant 0 \qquad i = 1, N_{beam} \quad k = 1,3 \qquad (42)$$

where i = number of the beam (i.th beam), N_{beam} = total number of beams, k = three critical region (left and right ends, and span) for shear design of beam.

The spacing between stirrups s, in the middle and the ends of the beam should be greater than the minimum spacing, $s_{\min}$ ($s_{\min}$ = 50 mm) for constructional requirements;

$$g_{29}(x) = \frac{s_{\min}}{s_{(i,k)}} - 1 \leqslant 0 \qquad k = 1, N_{beam} \quad j = 1,3 \qquad (43)$$

where i = number of the beam (i.th beam), N_{beam} = total number of beams, k = three critical region (left and right ends, and span) for shear design of beam.

At the left and right ends of beam members, the spacing between stirrups, s, should be less than maximum spacing limit of shear reinforcement for end regions of beam $s_{\max,end}$;

$$g_{30}(x) = \frac{s_{(i,j)}}{s_{\max,end}} - 1 \leqslant 0 \qquad i = 1, N_{beam} \quad k = 1,2 \qquad (44)$$

$$s_{\max,end} \to \min \begin{cases} \frac{h}{4} \\ 30\ \text{cm} \\ 24\phi_b \end{cases} \qquad (45)$$

where i = number of the beam (ith beam), N_{beam} = total number of beams, k = three critical regions (left and right ends and the span) for shear design of beam, h = the height of beam section, ϕ_b = the diameter of shear reinforcing bars (ties).

Along the span of a beam member, the spacing between stirrups s should be less than maximum spacing of the shear reinforcement, $s_{\max,middle}$;

$$g_{31}(x) = \frac{s_{(i,middle)}}{s_{\max,middle}} - 1 \leqslant 0 \qquad i = 1, N_{beam} \qquad (46)$$

$$s_{\max,middle} \to \min \begin{cases} \frac{h}{2} \\ 60\ \text{cm} \\ 30\frac{A_{sv}}{f_y} b_w \end{cases} \qquad (47)$$

where i = number of the beam (ith beam), N_{beam} = total number of beams, h = the height of beam section, b_w and h = the width and height of column section, A_{sv}= the area of shear reinforcement (tie), f_y = the yielding strength of reinforcing steel.

The area of shear reinforcement (ties) in beam sections (span and ends of beam), A_{sv}, should be greater than limitations on the minimum area of shear reinforcement, $A_{sv,\min}$;

$$g_{32}(x) = \frac{A_{sv,\min}}{A_{sv(i,k)}} - 1 \leqslant 0 \qquad i = 1, N_{beam} \quad k = 1,3 \qquad (48)$$

$$A_{sv,\min} = 0.062\sqrt{f_c} b_w h \frac{s}{f_y} \geqslant 0.35h\frac{s}{f_y} \qquad (49)$$

where i = number of the beam (ith beam), N_{beam} = total number of beams, k = three critical regions (left and right ends, and along the span) for shear design of a beam, f_c = the compressive strength of concrete,f_y = the yielding strength of reinforcing steel, b_w and h = the width and height of beam section, s = the spacing between stirrups (ties).

The width of beams, b_w, should be greater than allowable minimum width for beams, $b_{w,\min}$;

$$g_{33}(x) = \frac{b_{w,\min}}{b_{w(i)}} - 1 \leqslant 0 \qquad i = 1, N_{beam} \qquad (50)$$

$$b_{w,\min} \to \max \begin{cases} 25\ \text{cm} \\ \frac{L_{beam(i)}}{50} \end{cases} \qquad (51)$$

where i = number of the beam (ith beam), N_{beam} = total number of beams, L_{beam} = the length of beam.

The height of beams, h, should be greater than allowable minimum height for beams, $h_{,\min}$;

$$g_{34}(x) = \frac{h_{\min}}{h_{(i)}} - 1 \leqslant 0 \qquad i = 1, N_{beam} \qquad (52)$$

$$h_{\min} \to \begin{cases} \frac{L_{beam(i)}}{18.5} & \textit{for one end continuous} \\ \frac{L_{beam(i)}}{21} & \textit{for both end continuous} \end{cases} \qquad (53)$$

where i = number of the beam (ith beam), N_{beam} = total number of beams, L_{beam} = length of the beam.

2.3.3. Constraints for joints

At frame joints, the width of beams, b_w, should be smaller than the width of column, b, framed into the ends joint of beam;

$$g_{35}(x) = \frac{b_{w(i)}}{b_{(i)}} - 1 \leqslant 0 \qquad i = 1, N_{joint} \qquad (54)$$

where i = number of the joint (ith joint), N_{joint} = total number of joints.

The width of the top column, b, should be equal or smaller than the width of the bottom column at the column joints;

$$g_{36}(x) = \frac{b_{top\ column(i)}}{b_{bottom\ column(i)}} - 1 \leqslant 0 \qquad i = 1, N_{joint} \qquad (55)$$

where i = number of the joint (ith joint), N_{joint} = total number of joints.

The height of the top column, h, should be equal or smaller than the width of the bottom column at the column joints;

$$g_{37}(x) = \frac{h_{top\ column(i)}}{h_{bottom\ column(i)}} - 1 \leqslant 0 \qquad i = 1, N_{joint} \qquad (56)$$

where i = number of the joint (ith joint), N_{joint} = total number of joints.

The sum of nominal flexural strengths of columns framing into the joint, ΣM_{nc}, should be 1.2 times greater than the sum of nominal flexural strengths of beams framing into same joint, ΣM_{nb},

$$g_{38}(x) = \frac{\frac{6}{5}\Sigma M_{nb(i)}}{\Sigma M_{nc(i)}} - 1 \leqslant 0 \qquad i = 1, N_{joint} \qquad (57)$$

where i = number of the joint (ith joint), N_{joint} = total number of joints.

The relative story displacements, Δ_i, should be smaller than the allowable story drift, Δ_a, given in the design specification;

$$g_{39}(x) = \frac{\left(\frac{\Delta_i}{h_i}\right)}{\Delta_a} - 1 \leqslant 0 \qquad i = 1, N_{story} \tag{58}$$

where i = number of the story (ith story), N_{story} = total number of stories, h_i = the height of story from base level.

3. Harmony search method

The harmony search (HS) algorithm is used to determine the optimum solution of the design optimization problem described in the previous section. This algorithm is one of the recent meta-heuristic optimization techniques which is originated by Geem et al. [17–19] The harmony search algorithm only needs adjustment of a few parameters, consists of simple steps and has a fast convergence rate [20–28]. The algorithm is widely used to obtain the solution of optimum structural design problems [29–33].

The HS algorithm mimics the improvisation of music players for searching the better harmony. The various possible combinations of the musical pitches stored in the musicians' memory, when they compose a harmony. And musicians can find the pleasing harmony from their harmony memories. The players play any pitch within the possible range, and these sounds make one harmony vector in music improvisation. These harmony vectors are stored in each player's memory if all the pitches make a good harmony, and the possibility to make a better harmony is increased next time. In the adaptation of this musical improvisation process into the solution of optimization problems, each musician corresponds to a decision variable and possible notes correspond to the possible values of decision variables. Similarly in optimization problems, each decision variable initially chooses any value within the possible range, and these values of decision variables make one solution vector. If all the values of decision variables make a good solution, these solution vectors are stored in the memory. Thus, the possibility to make a good solution is also increased next time. For example, if the first musician plays {La} while second and third musicians play {Fa} and {Do} from their harmony memories or randomly, a new harmony is created {La, Fa, Do}. And if this new harmony is better than the existing worst harmony in the harmony memory (HM), the new harmony is included in the HM and the worst harmony is excluded from the HM. This procedure is repeated until a nice harmony is found. In engineering optimization problems, decision variables are replaced with musicians, and the values selected for variables are replaced with sound pitches of musicians. The steps of harmony search algorithm are as follows;

Step 1. Optimization problem and algorithm parameters are initialized.
Step 2. Harmony memory matrix (HM) is initialized.
Step 3. A new harmony from the HM is improvised.
Step 4. Harmony memory matrix is updated.
Step 5. Steps 3 and 4 are repeated until the termination criterion is satisfied.

These steps can be summarized in the following:

Step 1 – The discrete optimization problem is specified as follows:

$$\boldsymbol{Minimize}\ \boldsymbol{f}(\boldsymbol{x})\ \boldsymbol{s.t.}\ \boldsymbol{x}_i \quad \in \mathrm{X}_i, i = 1, 2, \ldots, \mathrm{N}$$

where X_i is the set of possible candidate values for each decision variable i. A design variable pool is constructed for each design variable. The number of solution vectors in harmony memory matrix (HMS), harmony memory considering rate (HMCR), pitch adjusting rate (PAR), and termination criterion (maximum number of search) are also decided in this step.

Step 2 – The "harmony memory" (HM) matrix is filled with randomly generated solution vectors using the values in the design pool and sorted by the values of the objective function, $f(x)$. The Harmony Memory is filled with as many solution vectors as harmony memory size (HMS). The harmony memory matrix has the following form:

$$[H] = \begin{bmatrix} x_1^1 & x_2^1 & \cdots & \cdots & x_{N-1}^1 & x_N^1 \\ x_1^2 & x_2^2 & \cdots & \cdots & x_{N-1}^2 & x_N^2 \\ \cdots & \cdots & \cdots & \cdots & \cdots & \cdots \\ \cdots & \cdots & \cdots & \cdots & \cdots & \cdots \\ x_1^{HMS-1} & x_2^{HMS-1} & \cdots & \cdots & x_{N-1}^{HMS-1} & x_N^{HMS-1} \\ x_1^{HMS} & x_2^{HMS} & \cdots & \cdots & x_{N-1}^{HMS} & x_N^{HMS} \end{bmatrix}$$

Table 5
Input data for two-bay six-story RC frame example.

Input datas for two-bay six-storey frame	
Compressive strength of concrete, f_c (N/mm^2)	20
Yielding stress of steel, f_y (N/mm^2)	415
Cover for beams, mm	25
Cover for columns, mm	25
Total number of beam groups	2
Total number of beams	12
Total number of column groups	3
Total number of columns	18
Lateral force for each story (kN)	10
Factored uniformly distributed load on beams (kN/m)	30
Harmony Search Algorithm Parameters	
HMS	45
HMCR	0.80
PAR	0.15
Max. iteration	100,000

Table 6
The optimum values of design variables for two-bay six-story RC frame.

Optimum design results for two-bay six-storey frame			
Design variables	Column Gr. 1	Column Gr. 2	Column Gr. 3
Optimum column design results			
Section number in design pool	(3)	(82)	(3)
The height of section, h (mm)	400	500	400
The height of section, b (mm)	350	300	350
The diameter of reinforcement bars, ϕ(mm)	14	14	14
The number of bars in x direction, n_1	2	3	2
The number of bars in y direction, n_2	1	2	1
Total reinforcement	10ϕ14	14ϕ14	10ϕ14

Design variables	Beam Gr. 1	Beam Gr. 2
Optimum beam design results		
x_1	25/40 (1)	25/40 (1)
x_2	2ϕ12 (1)	2ϕ12 (1)
x_3	2ϕ12 (1)	2ϕ12 (1)
x_4	1ϕ16 (4)	1ϕ16 (4)
x_5	1ϕ16 (4)	1ϕ14 (3)
x_6	1ϕ12 (2)	1ϕ12 (2)
x_7	1ϕ12 (2)	1ϕ12 (2)
x_8	1ϕ14 (3)	2ϕ14 (7)
x_9	*NNR** (1)	*NNR** (1)
x_{10}	1ϕ14 (3)	1ϕ16 (4)
x_{11}	*NNR** (1)	*NNR** (1)
x_{12}	*NNR** (1)	*NNR** (1)
x_{13}	*NNR** (1)	*NNR** (1)

*NNR**, no need reinforcement, (n_i), the sequence number of ith design variable in the design variable pool.

Each row of harmony memory matrix contains the values of design variables of the optimization problem and presents one feasible solution vector. The design variables are randomly selected from the design variable pool for that particular design variable. Hence, this memory matrix has N columns (*N; the total number of design variables*) and HMS rows (*HMS; harmony memory size*). These candidate solutions are sorted such that the objective function value corresponding to the first solution vector (first row) is the minimum or maximum.

Step 3 – A new harmony vector, $\boldsymbol{x}^{\prime}=(\boldsymbol{x}_1^{\prime},\boldsymbol{x}_2^{\prime},\boldsymbol{x}_3^{\prime},\ldots,\boldsymbol{x}_N^{\prime})$ is improvised. There are three rules to choose one value for each decision variable; harmony memory consideration rate (HMCR), pitch adjustment rate (PAR), and random selection.

In memory consideration, the value of the first decision variable $(x_1^{\prime})$ can be chosen from any discrete value in the specified HM range $\{\boldsymbol{x}_1^1,\boldsymbol{x}_1^2,\boldsymbol{x}_1^3,\ldots,\boldsymbol{x}_1^{HMS-1},\boldsymbol{x}_1^{HMS}\}$ with the probability of HMCR which varies between 0 and 1. Values of the other decision variables $(\boldsymbol{x}_i^{\prime})$ can be chosen in the same manner. However, there is still a chance where the new value can be randomly chosen from a set of entire possible values in the design variable pools with the probability of (1-HMCR). That is

$$x_i^{\prime} \leftarrow \begin{cases} x_i^{\prime} \in \{x_i^1, x_i^2, \ldots, x_i^{HMS}\} & \text{with probability } HMCR \\ x_i^{\prime} \in X_i & \text{with probability } (1-HMCR) \end{cases}$$

Any component of the new harmony vector, whose value was chosen from the HM, is then examined to determine whether it should be pitch-adjusted. This operation uses pitch adjusting parameter (PAR) that sets the rate of pitch-adjustment decision as follows:

$$\text{Pitch adjusting decision for } x_i^{\prime} \leftarrow \begin{cases} \text{Yes with probability } PAR \\ \text{No with probability } (1-PAR) \end{cases}$$

If the pitch adjustment decision for $\boldsymbol{x}_i^{\prime}$ is Yes, $\boldsymbol{x}_i^{\prime}$ is replaced with $\boldsymbol{x}_i(\boldsymbol{k})$ (the $\boldsymbol{k}$th element $\boldsymbol{X}_i$), and the pitch-adjusted value of $\boldsymbol{x}_i(\boldsymbol{k})$ becomes:

$$\boldsymbol{x}_i^{\prime} \leftarrow \boldsymbol{x}_i(k \pm 1)$$

The algorithm chooses -1 or 1 for the neighboring index m with the same probability.

Step 4 – After selecting the new values for each design variable the objective function value is calculated for the new feasible harmony vector. If the new Harmony vector is better than the worst harmony in the HM in terms of the

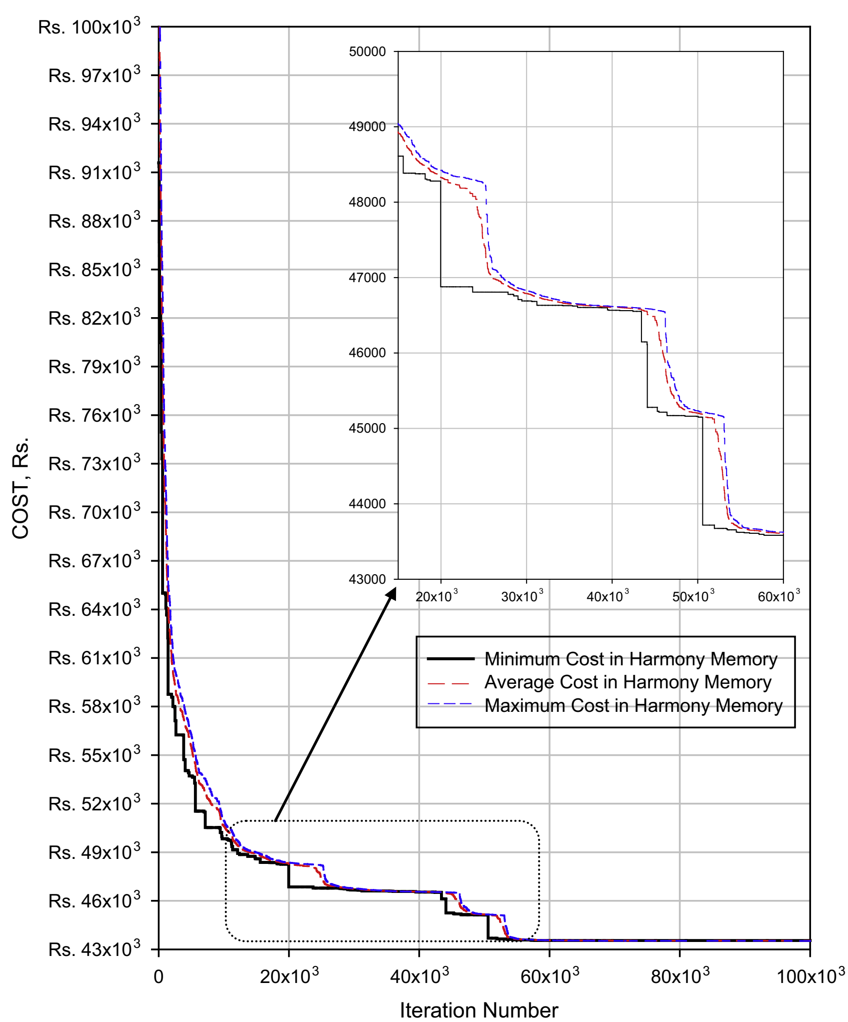

Fig. 5. The design history for two-bay six-storey RC plane frame (load combinations are not considered).

objective function value, the new harmony is included in the HM and the existing worst harmony is excluded from the HM. The HM is then sorted by the objective function value.

Step 5 – The computations are terminated when the termination criterion is satisfied. If not, Steps 3 and 4 are repeated. The termination criterion is selected large enough such that within this number of cycles no further improvement is observed in the objective function value.

3.1. Optimum design process with harmony search algorithm

The optimum design problem of RC frames necessitates finding the appropriate values of design variables described in Section 2.2 such that the design constraints given by the inequalities from (10)–(58) are satisfied and the objective function shown in Eq. (1) has the minimum value. The constraints used in this optimization problem are derived from ACI 318-05 [15] for *Special Seismic Moment Frames*. Also, the design calculations are performed for regulations of *Special Seismic Moment Frames* in ACI 318-05. The design variables are considered as two groups which are beam and column design variables. The values of beam and column design variables are to be selected from design pools shown in Tables 2 and 4 respectively. These tables contain discrete values for beam design variables due to practical reasons. For each column group, only one variable is assigned and this assigned value represents sequence number of the column section. This number varies between 1 and 219. The section properties of the column section (dimensions of column, diameter of reinforcing bars, and the number of reinforcing bars, etc.) are taken from the assigned row number. Hence, the solution technique to be adopted for the optimum design problem should be capable of obtaining the solutions of discrete programming problems efficiently.

Firstly, beams and columns are grouped to satisfy uniformity of members subject to close design forces and have similar behaviors according to their place in the frame and loading conditions. Then the total number of design variables is calculated by using the numbers of beam groups, column groups and bays in a frame

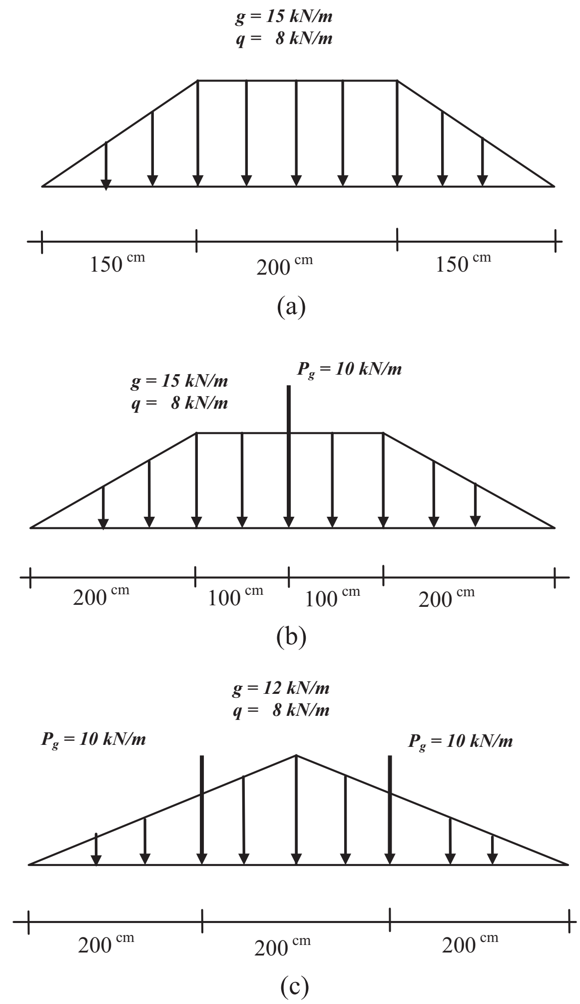

Fig. 7. Loading for five-bay five-storey frame (a) for beams at first and last spans, (b) for beams at second and fourth spans, and (c) for beams at third span.

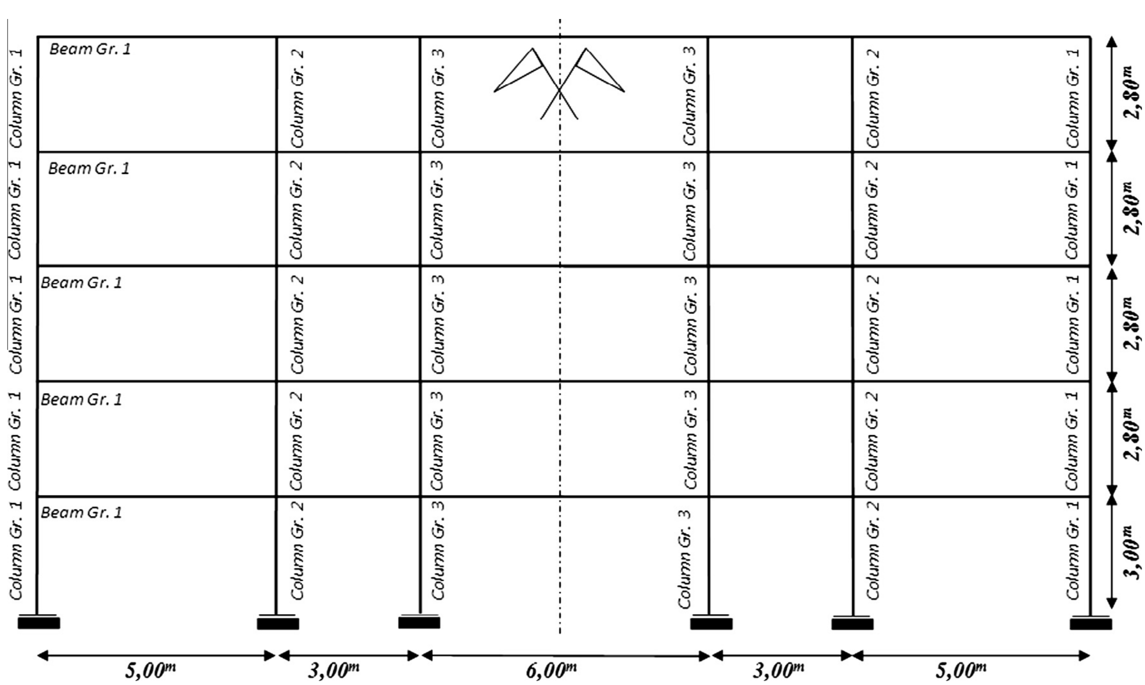

Fig. 6. Five-bay five-storey RC plane frame.

and the harmony search method parameters (HMS, HMCR, PAR, Max. iteration number) are selected. The randomly selected values are assigned for the design variables (beam and column) and the flexural and shear strength of beams are calculated for assigned section properties. Also, the column interaction diagrams are obtained for the assigned column section and material strengths and the flexural capacities of the column sections are determined for each load combination using the interaction diagram obtained for assigned column section. The constraints are checked to determine feasible design vectors and the cost (the objective function) of the frame is calculated. The Harmony Search Memory (HSM) is filled by these feasible design vectors randomly. In this phase, harmony memory consideration rate (HMCR), pitch adjustment rate (PAR), and random selection are used to determine the new values of design variables for each iteration. If the new design vector obtained in each iteration satisfies the constraints and the objective function value (the cost of frame) of the new design vector is better (smaller) than the worst objective function value in the Harmony Memory Matrix (HM), then the new design vector (design variables) is included in the HM and the existing worst harmony is excluded from the HM. The new HM with new design vector is then sorted by the objective function value (the cost of design vector). This process is repeated until the termination criterion is satisfied. At the end of this process, the first row of HM matrix gives the optimum design vector which includes the

Table 7
Input data for five-bay five-story RC frame.

Input datas for five-bay five-storey frame	
Compressive strength of concrete, f_c (N/mm^2)	30
Yielding stress of steel, f_y (N/mm^2)	400
Cover for beams (mm)	40
Cover for columns (mm)	50
Total number of beam groups	1
Total number of beams	25
Total number of column groups	3
Total number of columns	30
Seismic parameters	
Site class	C
S_s	0.50
S_1	0.20
F_a	1.20
F_v	1.60
R	8
I	1.0
T_L	6.0
I	1
S_{DS}	0.533
S_{D1}	0.2133
Harmony search algorithm parameters	
Case 1 – HMS = 40, HMCR = 0.80, PAR = 0.20	
Case 2 – HMS = 50, HMCR = 0.85, PAR = 0.15	
Max. iteration for each case	100,000

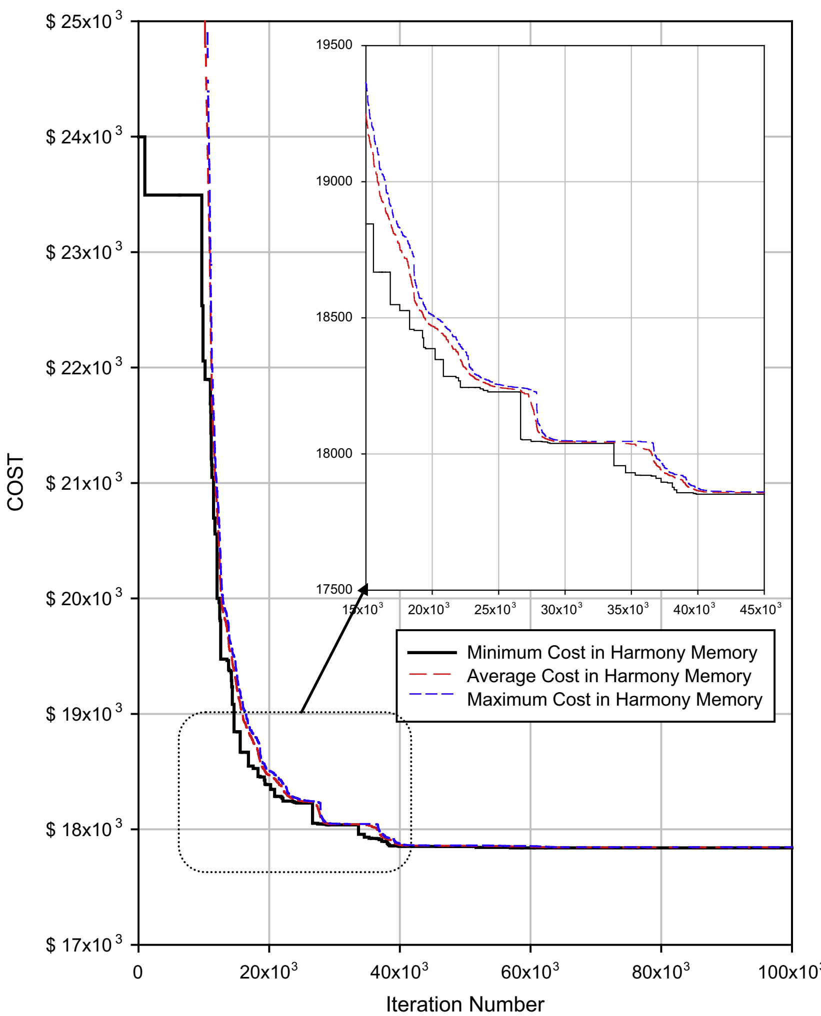

Fig. 8. The design memory history for the design of five-bay five-storey RC plane frame (for Case 1).

optimum values of design variables. The flowchart of this process is demonstrated in Fig. 3. The computer program in Fortran programming language was written to realize the explained procedure above. The results of the program were verified by considering solved examples from the literature.

4. Design examples

Two reinforced concrete plane frames are designed by the algorithm presented in order to demonstrate its efficiency. These are two-bay six-story and five-bay five-story frames. Design examples with different design variable numbers are selected to show the efficiency and the performance of the harmony search algorithm. Uniform gravity loads are assumed for dead load (D) and live load (L) and for seismic loading the lateral equivalent static earthquake loads which are calculated with respect to ASCE 7-05 are applied as joint loads. Six different factored load combinations are considered as suggested in ACI 318-05 design code. The design examples considered are solved several times using with different set of harmony search parameters and among the optimum frames obtained for each set, the best one is taken as the optimum design.

4.1. Two-bay six-story RC frame

A two-bay six-story RC frame given in Fig. 4 is designed by Rajeev and Krishnamoorthy [8], Camp et al. [9], Govindaraj and Ramasamy [13]. This frame has 12 beams and 18 columns, which are collected in five design groups: two groups for beams and three groups for columns. The compressive strength of concrete and yield strength of steel are 20 MPa and 415 MPa, respectively. The frame is loaded with the unfactored uniformly distributed load of 30 kN/m and the lateral force of 10 kN is applied at each story level. The load combinations are not considered in this practical optimum design problem. The unit costs of concrete, steel and formwork are 735 Rs./m^3, 7.1 Rs./kg and 54 Rs./m^2, respectively as given in [8]. The unit weight of concrete and steel is taken as 25 kN/m^3 and 78.5 kN/m^3, respectively. The input data of the frame is given in Table 5. In this design example, there are 29 design variables, 26 of which are beams (13 for each beam design group), the remaining 3 is for columns. The Harmony search method parameters HMS, HMCR, PAR are selected as 45, 0.80, and 0.15, respectively. The cost of optimum design is obtained as 43586.19 Rs. The optimum values for design variables obtained are given in Table 6. The harmony search algorithm design history is shown in Fig. 5. For this optimization problem, the minimum costs of optimum designs are obtained as 26,052 Rs., 24,959 Rs., and 22,966 Rs. by Rajeev and Krishnamoorthy [8], Camp et al. [9], Govindaraj, respectively which are half of what is attained in this study. However, it should be noticed that in these studies, shear design of concrete members are not considered. Furthermore the frame is not considered as Special Seismic Moment Frames and only simple flexural constraints are used to obtain the optimum design. The development lengths and hook lengths of reinforcement steel bars

Table 8
The optimum values of design variables of five-bay five-story RC frame example (for Case 1).

Optimum design results for five-bay five-storey frame			
Design variables	Column Gr. 1	Column Gr. 2	Column Gr. 3
Optimum column design results			
Section number in design pool	(29)	(51)	(117)
The height of section, h (mm)	300	350	300
The height of section, b (mm)	550	600	750
The diameter of reinforcement bars, ϕ (mm)	18	16	20
The number of bars in x dir., n_1	3	6	5
The number of bars in y dir., n_2	1	3	1
Total reinforcement	12ϕ18	22ϕ16	16ϕ20
Design variables			Beam Gr. 1
Optimum beam design results			
x_1			25/60 (10)
x_2			3ϕ12 (3)
x_3			2ϕ16 (4)
x_4			1ϕ14 (3)
x_5			4ϕ12 (12)
x_6			1ϕ14 (3)
x_7			1ϕ12 (2)
x_8			1ϕ14 (3)
x_9			6ϕ14 (29)
x_{10}			1ϕ14 (3)
x_{11}			1ϕ12 (2)
x_{12}			1ϕ14 (3)
x_{13}			4ϕ12 (12)
x_{14}			*NNR** (1)
x_{15}			1ϕ2 (8)
x_{16}			1ϕ16 (4)
x_{17}			*NNR** (1)
x_{18}			1ϕ16 (4)
x_{19}			1ϕ22 (10)
x_{20}			1ϕ16 (4)
x_{21}			1ϕ22 (10)
x_{22}			1ϕ16 (4)
x_{23}			*NNR** (1)
x_{24}			*NNR** (1)
x_{25}			1ϕ2 (8)

*NNR**, no need reinforcement, (n_i), the sequence number of ith design variable in the design variable pool.

Table 9
The optimum values of design variables for five-bay five-story RC frame (Case 2).

Optimum design results for five-bay five-storey frame			
Design variables	Column Gr. 1	Column Gr. 2	Column Gr. 3
Optimum column design results			
Section number in design pool	(29)	(121)	(117)
The height of section, h (mm)	300	300	350
The height of section, b (mm)	550	750	800
The diameter of reinforcement bars, ϕ (mm)	18	14	20
The number of bars in x dir., n_1	3	6	4
The number of bars in y dir., n_2	1	3	2
Total reinforcement	12ϕ18	22ϕ14	16ϕ20
Design variables			Beam Gr. 1
Optimum beam design results			
x_1			25/60 (10)
x_2			2ϕ12 (1)
x_3			3ϕ12 (3)
x_4			1ϕ18 (6)
x_5			1ϕ26 (16)
x_6			1ϕ18 (6)
x_7			1ϕ16 (4)
x_8			1ϕ18 (6)
x_9			5ϕ16 (31)
x_{10}			1ϕ18 (6)
x_{11}			1ϕ16 (4)
x_{12}			1ϕ18 (6)
x_{13}			1ϕ26 (16)
x_{14}			*NNR** (1)
x_{15}			2ϕ14 (7)
x_{16}			1ϕ12 (2)
x_{17}			*NNR** (1)
x_{18}			1ϕ12 (2)
x_{19}			3ϕ12 (9)
x_{20}			1ϕ12 (2)
x_{21}			3ϕ12 (9)
x_{22}			1ϕ12 (2)
x_{23}			*NNR** (1)
x_{24}			*NNR** (1)
x_{25}			2ϕ14 (7)

*NNR**, no need reinforcement, (n^i), the sequence number of ith design variable in the design variable pool.

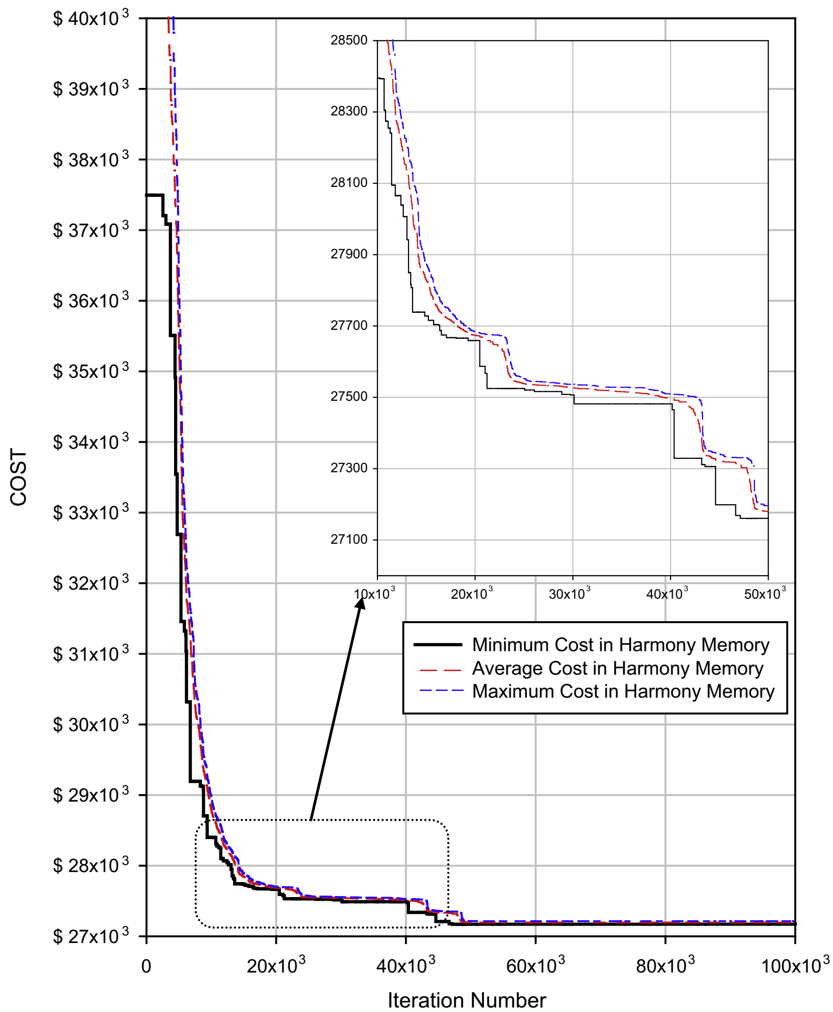

Fig. 9. The design history for five-bay five-storey RC plane frame (for Case 2).

Table 10
The details of material quantities and costs of the optimum designs for five-bay five-story RC frame.

Components	Case 1	Case 2
Concrete (m^3)	31.125	33.878
Beam	14.085	13.785
Column	17.040	20.093
Steel (kg)	7463.120	7277.768
Beam	2762.0	2795.3
Column	4701.119	4482.458
Formwork (m^2)	290.53	301.83
Beam	136.15	133.25
Column	154.38	168.580
Total cost ($)	17838.99	27159.44

in the concrete members and the other detailing issues are not considered in the calculation of the frame cost. Obviously the cost of optimum design found in this study is larger.

4.2. Five-bay five-story RC frame

The geometry and property group numbers of the five-bay five-story RC frame with 55 members is shown in Fig. 6 and the loading details is given Fig. 7. A frame has 25 beams and 30 columns, which are collected in five design groups: two groups for beams and three groups for columns. The compressive strength of concrete and yield strength of steel are 30 MPa and 400 MPa, respectively. The frame is loaded symmetrically and the loading details is given in Fig. 8. The load combinations that are given in the ACI 318-05 are considered in the optimum design. The seismic lateral forces are calculated according to ASCE 7-05 for given conditions. The unit weight of concrete and steel is taken as 25 kN/m^3 and 78.5 kN/m^3, respectively. The input data of the frame considered are given in Table 7. For this example, the frame is considered as symmetrical and there are 18 design variables, 15 of which are beams, the remaining 3 is for columns, in the optimum design problem. In this example, the example frame is analyzed for two cases of the unit costs of materials. The different unit costs are used to investigate the effect of unit material costs on the optimum designs. The unit costs of concrete, steel and formwork are taken 100 $/$m^3$, 1 $/kg and 25 $/$m^2$, respectively in the first case, and 110 $/$m^3$, 2.1 $/kg and 27 $/$m^2$, respectively in the second case. The Harmony search method parameters HMS, HMCR, PAR are selected as 40, 0.80, and 0.20, respectively for the first case, and 50, 0.85, and 0.15 for the second case. The minimum costs of optimum designs obtained for these two different cases are found to be $17838.99, and $27159.44. The quantities of concrete, formwork and steel obtained in these two cases are given in Table 10. The optimum values of the design variables in these two cases are listed in Tables 8 and 9. The design histories of the harmony search algorithm in these two cases are shown in Figs. 8 and 9.

The optimum unit quantities of concrete, formwork and steel obtained for two cases are given in Table 10. Optimum sections for the given unit cost of concrete, steel and concrete are obtained for both cases. In case two, due to the increased unit cost of steel, the optimum sections are selected with less steel usage. Even though there is a small increase in the amount of steel and a small decrease in the amount of concrete for the beams, the amount of steel is decreased and the amount of concrete is increased for the columns. Since more steel is used in the columns in comparison to the beams, the column and beam cross-section dimensions that provide the optimum quantity of concrete, steel, and formwork are impacted.

5. Conclusions

It is shown that the optimum design problem of reinforced concrete special seismic moment frames can be formulated according to the provisions of ACI 318-05 and its solution can be obtained by harmony search algorithm. The design algorithm developed do not only considers the design code requirements but also architectural and reinforcement detailing constraints such that once the optimum design is attained the solution is ready for practical application without need of any further process. The results obtained from the design examples optimized clearly indicates that the harmony search algorithm can efficiently be used in finding the optimum detailed of reinforced concrete frames. The optimum design algorithm arrives at rational and realistic design solutions that are ready for construction without further process. It is noticed that three main parameters of the harmony search algorithm namely, harmony memory size HMS, harmony memory considering rate HMCR, and the pitch adjustment rate PAR play an important role on the optimum designs obtained. It is also noticed that in the design example considered, the harmony search algorithm parameters, the high HMCR, especially from 0.70 to 0.95, the small PAR, generally from 0.15 to 0.20, and HMS, from 40 to 60, yielded good performance in the design optimization. Furthermore the unit costs of concrete, steel, and formwork play an important role in determining of the optimum column and beam section dimensions. It is also shown that optimum design of reinforced concrete frame without considering special seismic constraints yields unsafe design.

References

[1] Yang XS. Nature-inspired metaheuristic algorithms. Luniver Press; 2008.
[2] Yang XS. Engineering optimization: an introduction with metaheuristic applications. John Wiley; 2010.
[3] Saka MP, Dogan E. Recent developments in metaheuristic algorithms: a review. Comput Technol Rev 2012;5:31–78.
[4] Saka MP. Optimum design of skeletal structures: a review. In: Topping BHV, editor. Progress in civil and structural engineering computing. Saxe-Coburg Publications; 2003. p. 237–84 [chapter 10].
[5] Saka MP. Optimum design of steel frames using stochastic search techniques based on natural phenoma: a review. In: Topping BHV, editor. Civil engineering computations: tools and techniques. Saxe-Coburg Publications; 2007. p. 105–47 [chapter 6].
[6] Choi C, Kwak H. Optimum RC member design with predetermined discrete sections. J Struct Eng ASCE 1990;116(10):2634–55.
[7] Balling RJ, Yao X. Optimization of reinforced concrete frames. J Struct Eng ASCE 1997;123(2):193–202.
[8] Rajeev S, Krishnamoorthy CS. Genetic algorithm-based methodology for design optimization of reinforced concrete frames. Comput-Aided Civ Infrastruct Eng 1998;13(1):63–74.
[9] Camp CV, Pezeshk S, Hansson H. Flexural design of reinforced concrete frames using a genetic algorithm. J Struct Eng ASCE 2003;129(1):105–15.
[10] Lee C, Ahn J. Flexural design of reinforced concrete frames by genetic algorithm. J Struct Eng ASCE 2003;129(6):762–74.
[11] Kwak H-G, Kim J. Optimum design of reinforced concrete frames based on predetermined section database. Comput Aided Des 2008;40(3):396–408.
[12] Kwak H-G, Kim J. An integrated genetic algorithm complemented with direct search for optimum design of RC frames. Comput Aided Des 2009;41(7):490–500.
[13] Govindaraj V, Ramasamy JV. Optimum detailed design of reinforced concrete frames using genetic algorithms. Eng Optim 2007;39(4):471–94.
[14] Akin A, Saka MP. Optimum detailing design of reinforced concrete plane frames to ACI 318-05 using the harmony search algorithm. In: Topping BHV. (editor). Proceedings of the eleventh international conference on computational structures technology, paper 72. Dubrovnik, Croatia: Civil Comp Press; 4–7 September, 2012. doi:10.4203/ccp.99.72.
[15] ACI 318-05 Building code requirements for structural concrete and commentary. ACI Committee 318. Structural Building Code, American Concrete Institute, Farmington Hills, MI, USA; 2005.
[16] ASCE 7-05 Minimum design loads for buildings and other structures. Virginia, USA: American Society of Civil Engineers; 2005.
[17] Geem ZW, Kim J-H, Loganathan GV. A new heuristic optimization algorithm: Harmony search. Simulation 2001;76(2):60–8.
[18] Lee KS, Geem ZW. A new structural optimization method based on the harmony search algorithm. Comput Struct 2004;82(9-10):781–98.
[19] Lee KS, Geem ZW, Lee SH, Bae KW. The harmony search algorithm for discrete structural optimization. Eng Optim 2005;37(7):663–84.
[20] Lee KS, Geem ZW. A new meta-heuristic algorithm for continuous engineering optimization: harmony search theory and practice. Comput Methods Appl Mech Eng 2005;194:3902–33.
[21] Geem ZW, Lee KS, Tseng C-L. Harmony search for structural design. In: Proceedings of 2005 Genetic and Evolutionary Computation Conference (GECCO-2005). Washington, DC, USA; June 25–29 2005. p. 651–52.
[22] Geem ZW. Optimal cost design of water distribution networks using harmony search. Eng Optim 2006;38:259–80.
[23] Geem ZW. Novel derivative of harmony search algorithm for discrete design variables. Appl Math Comput 2008;199(1):223–30.
[24] Geem ZW, editor. Music-inspired harmony search algorithm. Springer; 2009.
[25] Geem ZW, editor. Recent advances in Harmony search algorithm. Springer; 2010.
[26] Geem ZW, editor. Harmony search algorithms for structural design optimization. Springer; 2010.
[27] Geem ZW. Harmony search algorithm. <www.harmonysearch.info> [accessed 04.05.11].
[28] Geem ZW, Roper WE. Various continuous Harmony search algorithms for web-based hydrologic parameter optimization. Math Modell Numer Optim 2010;1(3):226–31.
[29] Saka MP. Optimum geometry design of geodesic domes using harmony search algorithm. Adv Struct Eng 2007;10(6):595–606.
[30] Saka MP. Optimum design of steel swaying frames to BS 5950 using harmony search algorithm. J Constr Steel Res 2007;65(1):36–43.
[31] Erdal F, Saka MP. Effect of beam spacing in the harmony search based optimum design of grillages. Asian J Civ Eng 2008;9(3):215–28.
[32] Değertekin SO. Optimum design of steel frames using harmony search algorithm. Struct Multidisciplinary Optim 2008;36(4):393–401.
[33] Değertekin SO. Harmony search algorithm for optimum design of steel frame structures: a comparative study with other optimization methods. Struct Eng Mech 2008;29(4):391–410.