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# Optimum Detailing Design of Reinforced Concrete Plane Frames to ACI 318-05 using the Harmony Search Algorithm

A. Akin<sup>1</sup> and M.P. Saka<sup>2</sup>

<sup>1</sup> Department of Civil Engineering, Yuzuncu Yil University, Van, Turkey <sup>2</sup> Department of Civil Engineering, University of Bahrain, Isa Town, Bahrain

## 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 the 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.

**Keywords:** cost optimization, structural optimum design, harmony search algorithm, reinforced concrete structures, heuristics.

# **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, swarm intelligence or annealing process, etc.) that is automated by nature to achieve the task of optimization of its own [1,2]. 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 [3,4]. 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 frames and these algorithms require additional modifications to fit the discrete nature of structural design variables. Choi and Kwak [5] 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 [6]. Discrete variables as well as limits on the number of reinforcing bars and their topological arrangements are considered in their study. Rajeev and Krishnamoorthy [7] 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 crosssection dimensions. The allowable combinations of reinforcement bars for columns and beams were tabulated. Camp et al. [8] 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 [9] 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 [10] studied on optimum design of RC plane frames based on pre-determined section database. In their study, predetermined 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 [11]. Govindaraj and Ramasamy [12] 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.

This study deals with the problem of optimizing *Special Seismic Moment* reinforced concrete (RC) frames subject to ACI 318-05 [13] and ASCE 7-05 [14]. In the literature, there are a number of studies on optimization of RC members and frames. 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. The design algorithm presented in this study considers covers all required code provisions and obtains the optimum solution by using harmony search algorithm.

## 2 Modelling of Optimum Detailing Design Problem of Reinforced Concrete Plane Frames

Reinforced concrete special moment frames are used as part of seismic forceresisting 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. These moment-resisting frames are called "Special Moment Frames" because of the additional requirements. The optimum design algorithm developed for the RC special seismic moment frames obtains buildable optimum designs where the cross sections are selected from the design pool. 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. The design variables are categorized in two general groups i.e. 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 which considers shear and the seismic design constraints as well as flexural 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.

#### 2.1 **Objective Function**

The objective function is selected as the total cost of RC frame consisting of individual cost components of concrete, steel and formwork which is inclusive of material, fabrication, and labor. The objective function is expressed as,

Minimize

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

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 cost of concrete,  $C_c$  is computed from Eqn. (2) as;

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

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

$$V_{c} = \sum_{i=1}^{N_{col}} b_{i} d_{i} L_{column,i} + \sum_{j=1}^{N_{beam}} b_{w,j} h_{j} L_{clear \ beam,j}$$
(3)

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

The cost of reinforcing steel,  $C_s$  is computed from Eqn. (3) as;

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

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

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

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

The cost of formwork,  $C_f$  is computed from Eqn. (4) as;

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

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

$$A_{f} = \sum_{i=1}^{N_{col}} \{2(b_{i} + d_{i})L_{column,i} - (Area_{beams@joint,i})\} + \sum_{j=1}^{N_{beam}} \{(b_{w,j} + 2h_{j})L_{clear beam,j}\}$$
(7)

where, Area<sub>beams@joint,i</sub> = the cross-section area of beams connected to the  $i^{\text{th}}$  column at joint.

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 their design and different market prices for unit costs depending to their countries.

#### 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 crosssection 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 Figure 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 Eqn.8;

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

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.

Design Variat	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 *	1
X <sub>i,3</sub>	The area of the steel reinforcement that continues through the bottom of all the beams *	1
$X_{i,4}$ - $X_{i,k}$	The area of the top steel reinforcement at spans of beams*	<i>n</i> <sub>bay</sub>
$X_{i,k^{+}1}\text{-}X_{i,m}$	The area of the bottom steel reinforcement at spans of beams*	<i>n</i> <sub>bay</sub>
$X_{i,m+1}$ - $X_{i,n}$	The area of the top steel reinforcement at the supports*	$n_{bay}$ +1
$X_{i,n+1}$ - $X_{i,Nbdv}$	The area of the bottom steel reinforcement at supports*	$n_{bay}$ +1

*i*= number of beam group (*i*<sup>th</sup> beam group),  $n_{bay}$ ; number of spans (k = 3+  $n_{bay}$ , m= 3+2 $n_{bay}$ , n= 4+3 $n_{bay}$ , N<sub>bdy</sub>= 5+4 $n_{bay}$ )

\* 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 constructable reinforcement areas.

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

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 ( $n\phi d n$ ; 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 pre-assigned as design parameters at the beginning of the optimization 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.

#	$X_1$ (mm)	#	$X_2 - X_3$	#	$X_4 - X_{Nbdv}$
1	250/400	1	2¢12	1	NNR
2	250/450	2	2¢14	2	1¢12
3	250/500	3	3¢12	3	1¢14
4	250/550	4	2¢16	4	1¢16
5	250/600	5	3¢14	5	1¢18
6	300/400	6	3¢16	6	1¢20
7	300/450			7	1¢22
8	300/500			8	1¢24
9	300/550			9	1¢26
10	300/600			10	1¢28
11	300/650			11	1¢30
12	300/700			12	2¢12
13	350/500			13	2 <b>\oldsymbol{4}</b> 14
14	350/550			14	2¢16
15	350/600			15	2 <b>\$</b> 18
16	350/650			16	2¢20
17	350/700			17	2¢22
18	400/600				
19	400/650				
20	400/700			45	5 <b>φ</b> 18
21	400/750			46	5 <b>\$</b> 20
22	400/800			47	5¢22
23	450/700			48	5¢24
24	450/750			49	5 <b>¢</b> 26
25	450/800				
26	450/850				
27	450/900			55	6 <b>φ</b> 18
28	500/700			56	6ф20
29	500/750			57	6ф22
30	500/800			58	6ф24
31	500/850			59	6ф26
32	500/900			60	6ф28
				61	6¢30

\* NNR ; no need reinforcement

Table 2: Variable pool for beam design variables.



Figure 1: The reinforcement bar layout and the design variables for  $i^{th}$  beam group.

### 2.2.2 Column Design variables

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. 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.

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 Figure 2. The lower-upper bounds and increments of these variables are given in Table 3. The selected feasible column sections are given in Table 4. The compressive strength of concrete and the yield strength of steel are taken as 30MPa and 400MPa, respectively, and the cover of the concrete section is taken as 50mm 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.

Design Variables		Lower bound	Upper Bound	Increment (Step size)	Number of possible values in the range
X <sub>1c</sub>	<b>b</b> (mm)	300	500	50 mm	5
$X_{2c}$	<b>d</b> (mm)	400	1000	50 mm	13
$X_{3c}$	<b>\$</b> (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 3: Design variable bounds for RC column design examples.

#	d	b	ø	<b>n</b> 1	<i>n</i> <sub>2</sub>	P <sub>n,max</sub> (kN)	$P_{b}$ (kN)	M <sub>b</sub> (kNm)
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
			•				•	
45	550	500	14	$\dot{2}$	6	5307 57	204023	450 22
46	600	300	14	4	ĩ	3508.11	1374.17	294.71
47	600	300	18	3	2	3850,72	1368,77	362,88
48	600	350	16	3	1	4068,07	1596,43	346,04
01	700	350	16	7	2	5127 63	1034 52	520.61
91	700	350	18	7	1	5299.76	1954,52	537 78
93	700	350	16	7	3	5235 52	1931.12	559 51
94	700	350	24	3	1	5382 35	1909 13	587.06
156	800	450	24	4	5	8389,7	2814,16	1205,06
157	800	500	20	5	2	8006,54	3154,92	943,66
158	800	500	24	5	5	9272,95	3154,95	1299,31
159	850	350	14	7	1	5680,51	2371,26	664,1
•		•	•	•		•	·	
215		500	20	.7	2	9969 93	4041 03	1450.92
216	1000	500	$\frac{20}{20}$	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

Table 4: Design variables database for column sections.



Figure 2: The design variables for construction of column section database.

#### 2.3 Design Constraints

The design constraints considered in the optimum design problem are strength, serviceability, ductility and other side constraints that are implemented 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 number of reinforcing bars and other reinforcement requirements. The constraints are explained and expressed in a normalized form as given Table 5.

# **Constraints for Column Groups**

The maximum axial load capacity of columns, $P_{n,max}$ should be greater than the factored axial design load acting on the column section, $P_d$ ; 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. $M_d$ :	$g_{1}(x) = \frac{P_{d(i,j)}}{P_{n,\max(i,j)}} - 1 \le 0  i = 1, N_{col}  j = 1, 6  (10)$ where, $i = number$ of the column ( $i^{ih}$ column), $N_{col} = total$ number of columns, $j = load$ combination type. $g_{2}(x) = \frac{M_{d(i,j)}}{M_{n@P_{u(i,j)}}} - 1 \le 0  i = 1, N_{col}  j = 1, 6  (11)$ where, $i = number$ of the column ( $i^{th}$ column), $N_{col} = total$ number of columns, $j = load$ combination type.
The percentage of longitudinal reinforcement steel, $\rho$ , in a column section should be between minimum and maximum limits permitted by design specification ( $\rho_{min} = 0.01$ and $\rho_{max} = 0.06$ ); The width <i>b</i> and the height <i>d</i> of a column section should not be less than the minimum dimensions limit value given for columns ( <i>min. dimension</i> , $cd_{min} = 300mm$ );	$g_{3}(x) = \frac{\rho_{\min}}{\rho_{i}} - 1 \le 0  \text{and} $ $g_{4}(x) = \frac{\rho_{i}}{\rho_{\max}} - 1 \le 0  i = 1, N_{col}  (12)$ where, $i =$ number of the column ( $i^{\text{th}}$ column), $N_{col} =$ total number of columns. $g_{5}(x) = \frac{cd_{\min}}{b_{i}} - 1 \le 0$ $g_{6}(x) = \frac{cd_{\min}}{d_{i}} - 1 \le 0  i = 1, N_{col}  (13)$ where, $i =$ number of the column ( $i^{\text{th}}$ 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 \le 0 \qquad i = 1, N_{col} $ (14) where, $i$ = number of the column ( $i$ <sup>th</sup> column), $N_{col}$ = total number of columns.
The minimum diameter of longitudinal reinforcing bars, $\mathcal{O}$ , in a column section should be greater than minimum bar diameter, $\mathcal{O}_{min}$ , specified by design code;	$g_8(x) = \frac{\phi_{\min}}{\phi_i} - 1 \le 0 \qquad i = 1, N_{col} $ (15) where, $i$ = number of the column ( $i^{\text{th}}$ column), $N_{col}$ = total number of columns.
The total number of longitudinal reinforcing bars, <i>nrb</i> , 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 \le 0 \qquad i = 1, N_{col} $ (16) where, $i =$ number of the column ( $i^{\text{th}}$ column), $N_{col} =$ total number of columns.

	oups (com.)
The minimum and maximum clear spacings between	$g_{10}(x) = \frac{a_{\min}}{a_{i}} - 1 \le 0$ and
longitudinal bars, <i>a</i> , in a column section should be	$g_{11}(x) = \frac{a_i}{-1} - 1 \le 0 \qquad i = 1, N_{col} \tag{17}$
between minimum and	a <sub>max</sub>
maximum limits, $a_{min}$ and	where, $i =$ number of the column ( $i^{\text{th}}$ column), $N_{col} =$ total
$u_{max}$ , specified for detailing practice $(a_{1} = 50 \text{ mm} \text{ and}$	number of columns.
$a_{max}=150 \text{ mm}$ :	
The shear force capacity of	$V_{I(1)}$
column section, $\mathcal{O}V_n$ , should	$g_{12}(x) = \frac{d(i,j)}{dV} - 1 \le 0 \qquad i = 1, N_{col} \qquad j = 1, 6 \tag{18}$
be greater than applied	$\Psi_{I}^{(i,j)}$
factored design shear	where, $i =$ number of the column ( <i>i</i> column), $N_{col} =$ total number of columns, $i =$ load combination type
force, $V_d$ ;	number of columns, y foud combination type.
Also, the shear force capacity	$\min\{V_{e(k)}^{c} \mid V_{e(k)}^{b}\}$ (10)
of column section, $\mathcal{O}V_n$ ,	$g_{13}(x) = \frac{(19)}{\phi V_{(1)}} - 1 \le 0 \qquad k = 1, N_{column  group} $
should be greater than the	where $k =$ number of the column group ( $k^{\text{th}}$ column group)
minimum capacity snear forces $\min(V^c \mid V^b)$ based	$N_{col}$ = total number of column groups.
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,	
$V_e^b$ ;	
The factored design shear	$V_{d(i,j)} = V_{d(i,j)} = 1 < 0 \qquad i = 1 N \qquad i = 1 6 \qquad (20)$
force acting on column	$g_{14}(x) - \frac{1}{\phi V_{max(i,j)}} = 1 \le 0$ $l = 1, N_{col}$ $J = 1, 0$ (20)
section, $V_d$ , should be less than allowed maximum shear	where, $i =$ number of the column ( $i^{\text{th}}$ column), $N_{col} =$ total
force capacity $\partial V$ :	number of columns, $j = load$ combination type.
Toree cupacity, by max,	
The area of shear	Actoria
reinforcement (ties), $A_{sh}$ ,	$g_{15}(x) = \frac{s_{n,\text{min}}}{4} - 1 \le 0$ $k = 1, N_{column \in group}$ $j = 1, 3$
should be greater than	$A_{sh(i,j)}$
limitations on the minimum	$0.062\sqrt{f_c}bd\frac{s}{c} \ge 0.35d\frac{s}{c}$
area of shear reinforcement,	$J_y = J_y $ (21)
$A_{sh,min}$ ;	$A_{+-} \rightarrow \max \left\{ 0.3 sh \left(\frac{A_g}{c} - 1\right) \frac{f'_c}{c} \right\} $
	$A_{sh,min}$ , $A_{ch}$ , $A_{ch}$ , $f_y$
	$0.09 \text{ sh} \frac{f_c}{c}$
	$\int \int $
	where, $k =$ number of the column group ( $k^{\text{th}}$ column group),
	$N_{column group}$ = total number of column groups, $j$ = three (top,
	middle and bottom parts) shear design region of column, $f_c =$
	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

<b>Constraints for Column G</b>	roups <i>(Cont.)</i>
	$A_{sh}$ , $A_g$ = 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, $s_{min}$ ( $s_{min} = 50 \text{ mm}$ ) for constructional requirements;	$g_{16}(x) = \frac{s_{\min}}{s_{(i,j)}} - 1 \le 0 \qquad k = 1, N_{column \ group} \qquad j = 1,3  (22)$ where, $k$ = number of the column group ( $k^{th}$ 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, <i>s</i> , should be less than maximum spacing of shear reinforcement for end regions of column, <i>s</i> <sub>max,end</sub> ;	$g_{17}(x) = \frac{S_{(i,j)}}{S_{\max,end}} - 1 \le 0 \qquad k = 1, N_{column  group} \qquad j = 1, 2$ $s_{\max,end} \rightarrow \min \begin{cases} \frac{d}{4} \\ \frac{b}{4} \\ 6\phi_b \end{cases} $ (23)
	where, $k =$ number of the column group ( $k^{\text{th}}$ 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), $\mathcal{O}_b =$ the diameter of longitudinal reinforcing bars.
For the middle parts column members, the spacing between stirrups, <i>s</i> , should be	$g_{18}(x) = \frac{S_{(i,middle)}}{S_{\max,middle}} - 1 \le 0 \qquad k = 1, N_{column group}$
less than maximum spacing of shear reinforcement,	$s_{\max,middle} \to \min \begin{cases} 6\phi_b \\ 15 \ cm \end{cases}$ (24)
Smax,middle ,	where, $k =$ number of the column group ( $k^{\text{th}}$ column group), $N_{column group} =$ total number of column groups, $\mathcal{O}_b =$ the diameter of longitudinal reinforcing bars.
The length of top and bottom shear regions, $l_o$ , should be greater than allowable design	$g_{19}(x) = \frac{l_{0,\min}}{l_{0(i,j)}} - 1 \le 0 \qquad k = 1, N_{column \ group} \qquad j = 1, 2$
length, <i>l<sub>o,min</sub></i> ;	$l_{0,\min} \rightarrow \max \begin{cases} larger of b and d \\ 1/6 clear span of column \\ 45 cm \end{cases} $ (25)
	where, $k =$ number of the column group ( $k^{\text{th}}$ 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.

Table 5: Constraints are considered for the column members of the RC frame.

### 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 Table 6.

<b>Constraints for Beam Grou</b>	ups
For three critical sections (left end, middle part and right end) of each beam, the negative (with top steel in	$g_{20}(x) = \frac{M_{d(i,j,k,l)}}{M_{n(i,j,k,l)}} - 1 \le 0  i = 1, N_{beam}  j = 1,, 6  k = 1, 3  l = 1, 2$ (26)
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 $M_d$	where, $i =$ number of the beam ( $i^{\text{th}}$ 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	$g_{21}(x) = \frac{A_{s,\min(i,k,l)}}{A_{s(i,k,l)}} - 1 \le 0 \qquad i = 1, N_{beam}  k = 1, 3  l = 1, 2$
beam should satisfy the minimum and the maximum requirements permitted by	$g_{22}(x) = \frac{A_{s(i,k,l)}}{A_{s,\max(i,k,l)}} - 1 \le 0 \qquad i = 1, N_{beam}  k = 1, 3  l = 1, 2$
design specification;	$A_{s,\min} \to \max \begin{cases} 0.25 \frac{f_c'}{f_y} b_w h \\ 1.4 \frac{b_w h}{f} \end{cases}$
	(27)
	$A_{s,\max} = 0.025b_w h$ (for the total area of top and bottom flexural bars) where, $i =$ number of the beam ( $i^{\text{th}}$ 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	$g_{23}(x) = \frac{\frac{1}{2}M_{n,(i,k)}^{-}}{M_{n,(i,k)}^{+}} - 1 \le 0 \qquad i = 1, N_{beam}  k = 1, 2  (28)$
beam negative moment capacity $M_n^+$ , (i.e., associated with the top steel) at that end;	where, $i =$ number of the beam ( <i>i</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	$g_{24}(x) = \frac{\frac{1}{4}M_{n,(i,k)}^{-}}{M_{n,(i,middle)}^{+}} - 1 \le 0$ and

<b>Constraints for Beam Grou</b>	ups (Cont.)
positive flexural moment strengths at the ends of beams, $M_{n,-}^{-}$ and $M_{n,-}^{+}$ ;	$g_{25}(x) = \frac{\frac{1}{4}M_{n,(i,k)}^{+}}{M_{n,(i,middle)}^{+}} - 1 \le 0 \qquad i = 1, N_{beam}  k = 1, 2  (29)$
	where, $i =$ number of the beam ( <i>i</i> .th beam), $N_{beam} =$ total number of beams, $k =$ the left and right ends of beam.
The shear force capacity of three regions (left and right ends. and span) of beam.	$g_{26}(x) = \frac{V_{d(i,j,k)}}{\phi V_{n(i,j,k)}} - 1 \le 0 \qquad i = 1, N_{beam} \qquad j = 1, 6  k = 1, 3$
$\partial V_n$ , should be greater than	(30)
applied factored design shear force, $V_d$ ;	where, $i =$ number of the beam ( <i>i</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, $\mathcal{O}V_n$ , should be greater than	$g_{27}(x) = \frac{\max\left\{V_{e^{1}(i)}  V_{e^{1}(i)}\right\}}{\phi V_{n(i,k)}} - 1 \le 0 \qquad i = 1, N_{beam} \qquad k = 1,3$
the probable shear forces based on probable maximum	(31)
flexural strengths with the factored gravity loads at beam ends of beam, max $\{V_{el}, V_{e2}\}$ ;	where, $i =$ number of the beam ( <i>i</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	$g_{28}(x) = \frac{V_{d(i,k)}}{\phi V_{\max(i,k)}} - 1 \le 0 \qquad i = 1, N_{beam} \qquad k = 1, 3  (32)$
than allowed maximum shear force capacity, $OV_{max}$ ;	where, $i =$ number of the beam ( <i>i</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 <i>s</i> , in the middle and the ends of the beam should be greater	$g_{29}(x) = \frac{s_{\min}}{s_{(i,k)}} - 1 \le 0 \qquad k = 1, N_{beam} \qquad j = 1,3 \qquad (33)$
than the minimum spacing, $s_{min}=50mm$ for constructional requirements;	where, $i =$ number of the beam ( <i>i</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, <i>s</i> , should be	$g_{30}(x) = \frac{S_{(i,j)}}{S_{\max,end}} - 1 \le 0 \qquad i = 1, N_{beam}  k = 1, 2$
less than maximum spacing limit of shear reinforcement for end regions of beam	$s_{\max end} \to \min \left\{ \frac{h}{4} \right\} $ (34)
S <sub>max,end</sub> ;	$24\phi_b$
	where, $i =$ number of the beam ( $i^{\text{th}}$ 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, $\mathcal{O}_b =$ the diameter of shear reinforcing bars (ties).

Constraints for Beam Grou	ups (Cont.)
Along the span of a beam member, the spacing between stirrups <i>s</i> should be less than maximum spacing of the shear reinforcement, <i>s</i> <sub>max,middle</sub>	$g_{31}(x) = \frac{S_{(i,middle)}}{S_{\max,middle}} - 1 \le 0 \qquad i = 1, N_{beam}$ $\left[ \frac{h}{2} \right]$
	$s_{\max,middle} \to \min \begin{cases} 2 \\ 60 \ cm \\ 30 \frac{A_{sv}}{f_y} b_w \end{cases} $ (35)
	where, $i =$ number of the beam ( $i^{\text{th}}$ 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) $A_{rec}$ should be greater	$g_{32}(x) = \frac{A_{sv,\min}}{A_{sv(i,k)}} - 1 \le 0 \qquad i = 1, N_{beam}  k = 1, 3$
than limitations on the minimum area of shear	$A_{sv,\min} = 0.062\sqrt{f_c^{t}b_w h} \frac{z}{f_y} \ge 0.35h \frac{z}{f_y} $ (36) where, <i>i</i> = number of the beam ( <i>i</i> <sup>th</sup> beam), N <sub>beam</sub> = total number
remforcement, A <sub>sv,min</sub> ,	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	$g_{33}(x) = \frac{b_{w,\min}}{b_{w(i)}} - 1 \le 0 \qquad i = 1, N_{beam}$
for beams, $b_{w,min}$ ;	( 25 cm
	$b_{w,\min} \to \max \left\{ \frac{L_{beam(i)}}{50} \right\}$ (37)
	where, $i =$ number of the beam ( $i^{\text{th}}$ 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	$g_{34}(x) = \frac{h_{\min}}{h_{(i)}} - 1 \le 0$ $i = 1, N_{beam}$
for beams, $h_{,min}$ ;	$h_{\min} \rightarrow \begin{cases} \frac{L_{beam(i)}}{18.5} & \text{for one end continuous} \\ I \end{cases} $ (38)
	$\left(\frac{L_{beam(i)}}{21}  for both end continuous\right)$
	where, $i =$ number of the beam ( $i^{\text{un}}$ beam), $N_{beam} =$ total number of beams, $L_{beam} =$ length of the beam.

Table 6: Constraints are considered for the beam members of the RC frame.

#### **2.3.3** Constraints for Joints

<b>Constraints for Joints</b>	
At frame joints, the width of	$b_{w(i)}$ 1 < 0 $\cdot$ 1 M
beams, $b_w$ , should be smaller	$g_{35}(x) = \frac{0}{h} - 1 \le 0$ $i = 1, N_{joint}$ (39)
than the width of column, b,	$U_{(i)}$
framed into the ends joint of	where, $i =$ number of the joint ( <i>i</i> joint), $N_{joint} =$ total number of joints
The width of the top column	L
<i>b</i> should be equal or smaller	$\boldsymbol{g}_{\mathcal{N}}(\boldsymbol{x}) = \frac{D_{top \ column(i)}}{1 - 1} - 1 \le 0 \qquad i = 1, N $
than the width of the bottom	$b_{bottom \ column(i)}$
column at the column joints;	where, $i =$ number of the joint ( $i^{\text{th}}$ joint), $N_{ioint} =$ total number of
<b>,</b>	joints.
The height of the top column,	$h_{top, column(i)}$ 1 is a set 1 by
<i>h</i> , should be equal or smaller	$g_{37}(x) = \frac{1}{h} - 1 \le 0 \qquad i = 1, N_{joint} $ (41)
than the width of the bottom	$n_{bottom\ column(i)}$
column at the column joints;	where, $i =$ number of the joint ( $i^{m}$ joint), $N_{joint} =$ total number of joints
The sum of nominal flexural	6
strengths of columns framing	$\frac{1}{5}\sum M_{nb(i)}$
into the joint, $\Sigma M_{nc}$ , should	$g_{38}(x) = \frac{5}{\sum M} - 1 \le 0 \qquad i = 1, N_{joint} $ (42)
be 1.2 times greater than the	$\sum IVI nc(i)$
sum of nominal flexural	where, $i =$ number of the joint ( $i^{m}$ joint), $N_{joint} =$ total number of joints
strengths of beams framing	Joints.
into same joint, $\Sigma M_{nb}$ ,	
$\Delta_i$ , the relative displacements	$\left( \Delta_{i} \right)$
than the allowable story drift	$(\overline{h_i})$
$\Lambda_{2}$ given in the design	$g_{39}(x) = \frac{\langle \cdot , \cdot \rangle}{\Lambda} - 1 \le 0 \qquad i = 1, N_{story} $ $\tag{43}$
specification;	where $i =$ number of the story ( $i^{\text{th}}$ story) $N =$ total number
<b>. . . . . . . . . .</b>	of stories, $h_i$ = the height of story from base level.
	, , , , , , , , , , , , , , , , , , ,

Table 7: Constraints are considered for the RC frame joints.

## 3 Harmony Search Method

The Harmony Search (HS) method is a meta-heuristic optimization algorithm, which is proposed by Geem, Kim, and Loganathan [15]. The HS algorithm is inspired by the underlying principles of the musicians' improvisation of the perfect state of harmony. The HS algorithm mimics the improvisation of music players for searching a better harmony. The various possible combinations of the musical pitches stored in the musicians' memory, when they compose harmony. And the musicians can find the fantastic harmony from their harmony memories [16,17].

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. 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. 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. The optimization procedure of the Harmony Search meta-heuristic algorithm consists of Steps 1-5, as follows;

*Step 1-* The discrete optimization problem is specified as follows:

$$Minimize f(x) \quad s.t. \quad x_i \in X_i \quad i=1,2,\dots,N$$
(44)

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:

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,  $\mathbf{x}^{t} = (\mathbf{x}_{1}^{t}, \mathbf{x}_{2}^{t}, \mathbf{x}_{3}^{t}, \dots, \mathbf{x}_{N}^{t})$  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  $(x_1^{l})$  can be chosen from any discrete value in the specified HM range  $\{x_1^{l}, x_1^{2}, x_1^{3}, ..., x_l^{HMS-l}, x_l^{HMS-l}\}$  with the probability of HMCR which varies between 0 and 1. Values of the other decision variables  $(x_i^{l})$  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

$$x_{i}' \leftarrow \begin{cases} x_{i}' \in \{x_{i}^{1}, x_{i}^{2}, \dots, x_{i}^{HMS}\} \text{ with probability HMCR} \\ x_{i}' \in X_{i} \text{ with probability (1-HMCR)} \end{cases}$$

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:

Pitch adjusting decision for 
$$x'_{i} \leftarrow \begin{cases} Yes \text{ with probability } PAR \\ No \text{ with probability } (1 - PAR) \end{cases}$$

If the pitch adjustment decision for  $x_i^i$  is Yes,  $x_i^i$  is replaced with  $x_i(k)$  (the  $k^{th}$  element  $X_i$ ), and the pitch-adjusted value of  $x_i(k)$  becomes:

$$x_i^i \leftarrow x_i(k \pm 1)$$

The algorithm chooses - 1 or 1 for the neighbouring 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 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.

## 4 Design Example

Two-bay, six-storey RC plane frame is designed by the algorithm presented in order to demonstrate its efficiency. 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 Figure 3 is designed by Rajeev and Krishnamoorthy [7], Camp and Pezeshk [8], Govindaraj and Ramasamy [12]. 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 20MPa and 415MPa, respectively. The frame is loaded with the unfactored uniformly distributed load of 30kN/m and the lateral force of 10kN is applied at each story level. The load combinations are not considered in this practical optimum design problem to be able compare the results with the ones given in [7]. The unit costs of concrete, steel and formwork are 735 Rs./m<sup>3</sup>, 7.1 Rs./kg and 54 Rs./m<sup>2</sup>, respectively as given in [7]. The unit weight of concrete and steel is taken as 25kN/m<sup>3</sup> and 78.5kN/m<sup>3</sup>, respectively. The input data of the frame is given in Table 8. 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 obtained as 43586.19Rs. The optimum values for design variables obtained are given in Table 9. The harmony search algorithm design history is given in Figure 4. For this optimization problem, the minimum costs of optimum designs are obtained as 26052Rs., 24959Rs., and 22966Rs. by Rajeev [7], Camp [8], Govindaraj [12], respectively. In these studies, neither shear design constraints nor the requirements of Special Seismic Moment Frames are considered. Furthermore the development and hook lengths of reinforcement steel bars in the concrete members and the other detailing issues are not considered in the calculation of the frame cost. Therefore, the differences between obtained the optimum costs is based on different of design philosophies which exist between this study and the ones available literature.

Input datas for two-bay six-storey frame	
Compressive strength of concrete, $f_c$ (N/mm <sup>2</sup> )	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 8: Input data for two-bay six-story RC frame example.



Figure 3: Two-bay six-storey RC plane frame.



Figure 4: The design history for two-bay six-storey RC plane frame (load combinations are not considered.

## 5 Conclusions

The optimum design problem of reinforced concrete special seismic moment frames is formulated according to the provisions of ACI 318-05. In addition to the design constraints specified in the design code architectural and reinforcement detailing constraints are also considered in the design problem. It is shown that the harmony

Optimum design results for two-bay six-storey frame			
Optimum column design results			
Design Variables	Column	Column	Column
	Gr. 1	Gr. 2	Gr. 3
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, $\phi$ (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	10\phi14	14014	10\phi14
Optimum beam design results	·	•	·
Design Variables	Beam Gr. 1	Beam Gr. 2	
$\mathbf{x}_1$	25/40(1)	25/40(1)	
x <sub>2</sub>	2\operatorname{1} (1)	2\$\operatorname{12} (1)	
X <sub>3</sub>	2\operatorname{1} (1)	2\$\overline{12} (1)	
$\mathbf{x}_4$	1¢16 (4)	1016 (4)	
$\mathbf{X}_5$	1¢16 (4)	1\otop14 (3)	
x <sub>6</sub>	1\otimes12 (2)	$1\phi12$ (2)	
X <sub>7</sub>	1\otimes12 (2)	1012 (2)	
X <sub>8</sub>	$1\phi 14_{*}(3)$	$2\phi 14_{*}(7)$	
X9	$NNR^{+}(1)$	$NNR^{+}(1)$	
X <sub>10</sub>	$1\phi 14_{1}(3)$	$1\phi 16_{*}$ (4)	
X <sub>11</sub>	$NNR^{*}(1)$	$NNR_{*}^{*}$ (1)	
x <sub>12</sub>	$NNR^{*}$ (1)	$NNR_{*}^{+}(1)$	
X <sub>13</sub>	$NNR^{*}$ (1)	$NNR^{*}$ (1)	

 $NNR^*$ , no need reinforcement, ( $n_i$ ), the sequence number of  $i^{th}$  design variable in the design variable pool.

Table 9: The optimum values of design variables for two-bay six-story RC frame.

search algorithm can efficiently be used to determine the solution of this optimum design problem. 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. The results obtained from the design examples are shown that harmony search method is a reliable and robust technique that can be effectively used in finding the optimum detailed design of reinforced concrete frames. The optimum design algorithm developed arrives at rational and realistic design solutions that are directly constructible. It is also shown that optimum design opt reinforced concrete frame without considering special seismic constraints yields unsafe design.

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