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Design Optimization of Semi-Rigidly Connected Steel Frames Using Harmony Search Algorithm

Mohammed Arafa[1], Ashraf Khalifa[2], Mamoun Alqedra[3]

[1]*Associate Professor, The Islamic University of Gaza, P. O. Box 108, Palestine, marafa@iugaza.edu.ps*
[2] *M.Sc., The Islamic University of Gaza, Palestine, khalifa.tec@gmail.com*
[3]*Associate. Professor, The Islamic University of Gaza, Palestine, malqedra@iugaza.edu.ps*

Abstract—In this paper, a design optimization algorithm is presented for non-linear steel frames with semi-rigid beam-column connections using harmony search algorithm. The design algorithm obtains the minimum steel weight by selecting from a standard set of steel sections. Strength constraints of American Institute of Steel Construction - Load and Resistance Factor Design (AISC-LRFD) specification, displacement, deflection, size constraint and lateral torsional bulking are imposed on frames. Harmony search (HS) is a recently developed meta-heuristic search algorithm which is based on the analogy between the natural musical performance and searching the solutions to optimization problems. The HS algorithm accounts for the effect of connections' flexibility and the geometric non-linearity of the members. The Frye–Morris polynomial model is used for modeling semi-rigid connections. Two design examples with extended end plate without column stiffeners are presented to demonstrate the application and validity of the algorithm.

Index Terms— Optimum design; Harmony search algorithm, Genetic algorithm, Semi-rigid connections; Frye and Morris model.

I INTRODUCTION

Structural design optimization of steel frames generally requires the selection of steel sections for the beams and columns from a discrete set of practically available steel section tables. This selection is carried out in such a way that the steel frame has the minimum weight, while the design is limited by constraints such as the choice of material, the feasible strength, displacements, deflection, size constraints, lateral torsional buckling and the true behavior of beam-to-column connections. The design algorithm aims at obtaining minimum steel weight frames by selecting a standard set of steel sections such as AISC wide-flange shapes [1].

Computer-aided optimization is traditionally used to obtain more economical designs since the 1970s [2]; [3] and [4]. Numerous algorithms have been developed for accomplishing the optimization problems in the last four decades. Presently engineers and designers are compelled to achieve more economical designs and to search or develop more effective optimization techniques; this is why heuristic search methods emerged in the first half of 1990s [5], [6] and [7].

In recent years, structural optimization witnessed the emergence of novel and innovative stochastic search techniques. These stochastic search techniques make use of the ideas taken from nature and do not suffer the discrepancies of mathematical programming based optimum design methods. Meta-heuristic algorithms typically intend to find a suitable solution for any optimization problem by 'trial-and-error' in a reasonable amount of computational time. During the last few decades, several meta-heuristic algorithms have been proposed. These algorithms include: Genetic algorithms (GAs), which are search algorithms based on natural selection and the mechanisms of population genetics. The theory was proposed by Holland [8] and further developed by Goldberg [9] and others. Genetic Programming, which is an extension of genetic algorithms, was developed by Koza [10]. He suggested that the desired program should itself evolve during the evolution process. Evolutionary programming, which was originally developed by (Fogel et al.) [11], described the evolution of finite state machines to solve prediction tasks. Evolution strategies, were developed to solve parameter optimization problems by (Schwefel et al.) [12], in which a deterministic ranking is used to select a basic set of solutions for a new trial [13].

Ant colony optimization (ACO), which was first formulated by Dorigo [14] and further developed by other pioneers [15] and [16]. This algorithm was obtained from the pheromone trails of ants. Particle Swarm Optimization (PSO), was developed by Kennedy and Eberhart [17], inspired by the swarm behavior of fish and bird schooling in nature [18], [19] and [20]. Bee Algorithms (ABC), is developed by Karaboga and Basturk [21] for solving optimization problems. Not long ago, a large number of optimum structural design algorithms have been developed which are relying on these effective, powerful and novel techniques such as Genetic algorithm based optimum design of nonlinear planar steel frames with various semi-rigid connections by Kameshki and Saka [22], design of steel frames using ant colony optimization by Camp, et. al. [23] and optimum design of cellular beams using harmony search and particle swarm optimizers by Erdal, et. al. [24].

Geem and Kim [25] developed a New Harmony search (HS) meta-heuristic algorithm that was conceptualized using the musical process of searching for a perfect state of harmony. The harmony in music is analogous to the optimization solution vector, and the musician's improvisations are analogous to local and global search schemes in optimization techniques. The HS algorithm does not require initial values for the decision variables. Furthermore, instead of a gradient search, the HS algorithm uses a stochastic random search that is based on the harmony memory considering rate and the pitch adjusting rate (defined in harmony search meta-heuristic algorithm section) so that derivative information is unnecessary. Compared to earlier meta-heuristic optimization algorithms, the HS algorithm imposes fewer mathematical requirements and can be easily adopted for various types of engineering optimization problems.

The main differences between HS and GA are summarized as: (i) HS generates a new design considering all existing designs, while GA generates a new design from a couple of chosen parents by exchanging the artificial genes; (ii) HS takes into account each design variable independently. On the other hand, GA considers design variables depending upon building block theory. (iii) HS does not code the parameters, whereas GA codes the parameters. That is, HS uses real value scheme, while GA uses binary scheme (0 and 1).

The current study develops an algorithm to obtain the optimum design of steel frames with semi-rigid beam-column connections using Harmony Search (HS) technique. The design optimization problem was formulated to obtain the minimum steel frame weight. The AISC-LRFD specifications [1] were imposed on the strength, displacement, deflection, size constraints. The Frye and Morris polynomial model is introduced to model the semi-rigid connections. To demonstrate the application of the developed algorithm, two steel frames with extended end plate moment connections are presented.

II DESIGN OPTIMIZATION PROBLEM.

The formulation of the current problem as an optimization problem is carried out by identifying the design variables, objective function, penalized objective function and penalty function.

A Design variables.

Structural design optimization of steel frames generally requires selection of steel sections for its beams and columns from a discrete set of practically available steel section tables. The design algorithm aims to obtain the minimum steel weight of frames by selecting a standard set of steel sections. The current study utilizes the AISC wide-flange shapes from W40 to W8 as the design variables of the optimization problem. These sections are considered the most practical sections for steel beams and columns.

B The objective function.

The adopted optimization problem of the design of steel frames is to minimize the overall steel weight. The objective function of the minimization problem is formulated as follows:

$$Minimize\ W(x) = \sum_{i=1}^{ng} A_i \rho_i L_i \qquad (1)$$

In Equation 1, $W(x)$ is total weight of the members, ng is total numbers of groups in the frame, A_i is cross-sectional area of member, ρ_i and L_i are density and length of member i.

C Penalized objective function.

In order to assess the fitness of a trial design and determine its distance from the global optimum, the eventual constraint violation should be computed by means of a penalty function. The penalty function consists of a series of geometric constraints corresponding to the dimensions and shape of the cross sections, and a series of constraints related to the deflection and internal forces of the members of the structure. Thus, the penalty will be proportional to constraint violations, and the best design will have the minimum weight with no penalty. There are several studies devoted to the selection of penalty functions [**26**], [**27**] and [**28**]. In this study, the penalized objective function $\varphi(x)$ is applied and written for American Institute of Steel Construction Load and Resistance Factor Design (AISC-LRFD) code as follows [**1**]:

$$\varphi(x) = W(x).(1+\gamma.V)^{\varepsilon} \qquad (2)$$

Where, $\varphi(x)$ = Penalized Objective Function, γ = Penalty constant, V = Constraint violation function and ε = Penalty function exponent. In this study γ = 1.0, ε = 2.0 are considered [**23**].

D Penalty function.

The constraints of the current optimization problem comprise displacement constraints, size constraints, deflection constraints and strength constraints. Therefore, the constraint violation function of the optimization problem is expressed as:

$$V = \sum_{i=1}^{N_{jt}} V_i^{td} + \sum_{i=1}^{N_s} V_i^{id} + \sum_{i=1}^{N_{cl}} V_i^{sc} + \sum_{i=1}^{N_f} V_i^{sb} + \sum_{i=1}^{N_f} V_i^{db} + \sum_{i=1}^{N_c} V_i^{I} \qquad (3)$$

Where, V_i^{td} is constraint violations for top-storey displacement, V_i^{id} is constraint violations for inter-storey displacement, V_i^{sc} and V_i^{sb} are constraint violations for size constraints of column and beam, respectively, V_i^{db} is constraint violations for beam deflection and V_i^{I} is the interaction formulas of the LRFD specification; N_j^t = number of joints in the top storey. N_s and N_c are number of storeys except the top storey and number of beam columns, respectively. N_{cl} is the total number of columns in the frame except the ones at the bottom floor. N_f = is the number of storeys.

The computation of the penalty function of these constraints is illustrated below:

The penalty may be expressed as,

$$V_i = \begin{cases} 0 & if\ \ \lambda_i \le 0 \\ \lambda_i & if\ \ \lambda_i > 0 \end{cases} \qquad (4)$$

The displacement constraints are,

$$\lambda_i^{td} = \frac{d_t}{d_t^u} - 1.0 \le 0 \quad where\ i = 1, \dots\dots N_{jt} \qquad (5)$$

$$\lambda_i^{id} = \frac{d_i}{d_i^u} - 1.0 \le 0 \quad where\ i = 1, \dots\dots N_s \qquad (6)$$

Where, d_t: maximum displacement in the top storey, d_t^u: allowable top storey displacement (max height/300), d_i : interstorey displacement in storey i, d_i^u: allowable interstorey displacement (storey height/300).

The size constraint is given as follows,

$$\lambda_i^{sc} = \frac{d_{un}}{d_{bn}} - 1.0 \le 0 \quad where\ i = 1, \dots\dots N_{cl} \qquad (7)$$

$$\lambda_i^{sb} = \frac{d_{bf}}{d_{bc}} - 1.0 \le 0 \quad where\ i = 1, \dots\dots N_f \qquad (8)$$

Where, d_{un} and d_{bn} are depths of steel sections selected for upper and lower floor columns, d_{bf}, d_{bc} are the width of beam flange and column flange respectively.

The deflection control for each beam is given as follows,

$$\lambda_i^{db} = \frac{d_{db}}{d_{du}} - 1.0 \leq 0 \quad where\ i = 1, \ldots \ldots . N_f \qquad (9)$$

Where, d_{db}: maximum deflection for each beam, d_{du}: allowable floor girder deflection for unfactored imposed load ≤ L/360.

For members subjected to bending moment and axial force, the adopted strength constraints based on [**1**] are expressed as follows,

$$for \quad \frac{P_u}{\emptyset_c\, P_n} \geq 0.20$$

$$\lambda_i^I = \left(\frac{P_u}{\phi_c\, P_n}\right) + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b\, M_{nx}} + \frac{M_{uy}}{\phi_b\, M_{ny}}\right) - 1.0 \leq 0$$

$$where\ i = 1, \ldots . N_c \qquad (10)$$

$$for \quad \frac{P_u}{\emptyset_c\, P_n} < 0.20$$

$$\lambda_i^I = \left(\frac{P_u}{2\phi_c\, P_n}\right) + \left(\frac{M_{ux}}{\phi_b\, M_{nx}} + \frac{M_{uy}}{\phi_b\, M_{ny}}\right) - 1.0 \leq 0$$

$$where\ i = 1, \ldots . N_c N_c \qquad (11)$$

Where, P_u = factored applied compression load, P_n = nominal axial strength (compression), M_{ux}= factored applied flexural moment about the major axis, M_{uy} = factored applied flexural moment about the minor axis, M_{nx} = nominal flexural strength about the major axis, M_{ny} = nominal flexural strength about the minor axis (for two-dimensional frames, M_{uy} = 0), φ_c= resistance factor for compression (equal 0.90), φ_b = flexural resistance factor (equal 0.90).

A The nominal compressive strength of a member.

$$P_n = A_g . F_{cr} \qquad (12)$$

$$F_e = \frac{\pi^2 . E}{\left(\frac{KL}{r}\right)^2} \qquad (13)$$

$$F_{cr} = 0.658^{\,(F_y/F_e)}\, F_y \qquad when, \frac{F_y}{F_e} \leq 2.25 \qquad (14)$$

$$F_{cr} = 0.877\, F_e \qquad when, \frac{F_y}{F_e} > 2.25 \qquad (15)$$

Where, P_n = nominal axial strength (compression), A_g = cross-sectional area of member, F_{cr} = critical compressive stress, F_e = Euler stress, F_y = yield strength of steel, E = modulus of elasticity, K = effective-length factor, L = member length, r = governing radius of gyration. The effective length factor K, for an unbraced frame is calculated from the following approximated equation taken from [**29**]. The out-of-plane effective length factor for each column member is specified to be K_y = 1.0, while that for each beam member is specified to be K_y = L/6 (i.e., floor stringers at L/6 points of the span). The length of the unbraced compression flange for each column member is calculated during the design process, while that for each beam member is specified to be L/6 of the span length.

$$K = \sqrt{\frac{1.6 G_A G_B + 4.0(G_A + G_B) + 7.50}{G_A + G_B + 7.50}} \qquad (16)$$

Where, subscripts A and B denote the two ends of the column under consideration. The restraint factor G is stated as

$$G = \frac{\sum(I_c / L_c)}{\sum(I_B / L_B)} \qquad (17)$$

Where, I_c is the moment of inertia and L_c is the unsupported length of a column section; I_B is the moment of inertia and L_B is unsupported length of a beam section. Σ indicates a summation for all members connected to that joint (A or B) and lying in the plane of buckling of the column under consideration.

B The nominal flexural strength of a member.

Design strength of beams is $\varphi_b M_n$. As long as $\lambda \leq \lambda_p$, the M_n is equal to M_p and the shape is compact. The plastic moment M_p is that calculated from the following equation.

$$M_n = M_P = F_y . Z_x \qquad (18)$$

Where, M_n = nominal flexural strength, M_p = plastic moment, F_y = yield stress of steel, Z = the plastic section modulus, λ_p = slenderness parameter to attain M_p. φ_b =flexural resistance factor (equal 0.90). Details of the formulations are given in the [**1**].

III Connection Modeling and Analysis of Steel Frames.

The modeling of beam – column connection and steel frame members has been demonstrated by ANSYS software. The beams and columns of the frame were modeled using BEAM3, from ANSYS library elements. BEAM3 is a uniaxial element with tension, compression, and bending capabilities. The extended end plate connections without column stiffeners were simulated using a non-linear spring element, COMBIN39. It is a unidirectional element with nonlinear generalized force (moment) – deflection (rotation) capabilities that can be used in any analysis.

In the present study, the non-linear properties of the extended end plate connections were modeled using Frye-Morris polynomial model [**30**] because of its simple implementation. This model has the general form:

$$\theta_r = c_1 (kM)^1 + c_2 (kM)^3 + c_3 (kM)^5 \qquad (19)$$

Where, θ_r is a rotation (rad x10^{-3}), M is a moment connection (*Kip.in*), k is a standardization constant which depends upon the connection type and geometry; c_1, c_2, c_3 are the curve fitting constants. The values of these constants are given in Table 1 [**31**].

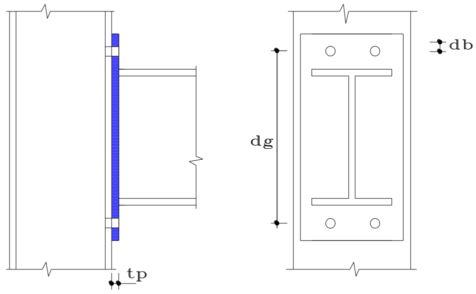
IV Design Optimization Using Harmony Search Algorithm.

Harmony Search technique (HS) was proposed by (Geem et al.) [25], [32], [33], [34] and [35] for solving combinatorial optimization problems. This approach is based on the musical performance process that takes place when a musician searches for a better state of harmony. A brief description of the implementation steps of the HS technique is presented in the following subsections:

A Initialize the harmony search parameters.

TABLE 1

Curve fitting constants and standardization constant for Frye-Morris Polynomial model.

Extended end plate without column stiffeners.	**Curve Fitting Constants Unit (inch)**	**Standardization Constant Unit (inch)**
	$c_1 = 1.83 \times 10^{-3}$ $c_2 = 1.04 \times 10^{-4}$ $c_3 = 6.38 \times 10^{-6}$	$k = dg^{-2.4}\, tp^{-0.4}\, db^{-1.5}$

The HS technique comprises several parameters to identify an algorithm which better represents a specific problem. These parameters comprise harmony memory (*HM*) matrix, harmony memory size (*HMS*), harmony memory consideration rate (*HMCR*), pitch adjusting rate (*PAR*), random uniformly distribution (*rand*), design variables (X_{sl}) and maximum iteration (Max_{iter}).

B Initialize harmony memory matrix.

In this step the harmony memory (HM) matrix is initialized by random selection of design variables from the adopted steel section list. The random selection is performed by using the interval [0, 1].The HM matrix can be represented as shown below:-

$$HM = \begin{bmatrix} x_1^1 & x_2^1 & \cdots & x_{ng-1}^1 & x_{ng}^1 \\ x_1^2 & x_2^2 & \cdots & x_{ng-1}^2 & x_{ng}^2 \\ \vdots & \vdots & \vdots & \vdots & \vdots \\ x_1^{HMS-1} & x_2^{HMS-1} & \cdots & x_{ng-1}^{HMS-1} & x_{ng}^{HMS-1} \\ x_1^{HMS} & x_2^{HMS} & \cdots & x_{ng-1}^{HMS} & x_{ng}^{HMS} \end{bmatrix} \begin{matrix} \rightarrow & \varphi(x^1) \\ \rightarrow & \varphi(x^2) \\ \rightarrow & \vdots \\ \rightarrow & \varphi(x^{HMS-1}) \\ \rightarrow & \varphi(x^{HMS}) \end{matrix} \qquad (20)$$

Where, x_i^1, x_i^2,…, x_i^{HMS-1},x_i^{HMS} and $\varphi(x^1)$,$\varphi(x^2)$,…, $\varphi(x^{HMS-1})$, $\varphi(x^{HMS})$ are design variables and the corresponding unconstrained objective function value, respectively.

C Generating a new harmony vector.

A new Harmony x_i^{new} is improvised from either the *HM* or design variables (X_{sl}). Three rules are applied for the generation of the new harmony. These are *HMCR*, *PAR* and *rand*.

$$x_i^{new} \leftarrow \begin{cases} x_i^{new} \in \{x_i^1, x_i^2, \ldots\ldots, x_i^{HMS-1}, x_i^{HMS}\} \\ \qquad if\ rand \le HMCR \\ x_i^{new} \in \quad x_{sl} \\ \qquad if\ rand > HMCR \end{cases} \qquad (21)$$

At first, a random number (*rand*) is generated, if this random number is equal or less than the *HMCR* value, x_i^{new} is selected from the current values stored in the i^{th} column of *HM*. If *rand* is higher than *HMCR*, x_i^{new} is selected from the design variables (X_{sl}).

Any design variable of the new harmony, x_i^{new} which obtained by the memory consideration is examined to determine whether it is pitch-adjusted or not. Pitch adjustment rate (*PAR*) which investigates better design in the neighboring of the current design as follow:

$$x_i^{new} \leftarrow \begin{cases} Yes, & if\ rand \le PAR \\ No, & if\ rand > PAR \end{cases} \qquad (22)$$

A random number (*rand*) is generated for x_i^{new}, if this random number is equal or less than the *PAR*, x_i^{new} is replaced with its neighboring section in the design variables (X_{sl}). If rand is higher than *PAR*, x_i^{new} remains the same. *HMCR* and *PAR* parameters are introduced to allow the solution to escape from local optima and to improve the global optimum prediction of the HS algorithm [**25**] and [**32**] .

D Update the harmony memory.

If the new harmony vector is better than the worst harmony in the *HM*, judged by objective function value, the new harmony is included in the *HM* and the existing worst harmony is excluded from the *HM*.

E Termination criteria.

If the termination criterion (Max_{iter}) is reached, computation is stopped. Otherwise, Steps C and D are repeated.

V Harmony Search Based Structural Optimization and Design Procedure.

Figure 1 shows the detailed procedure of HS algorithm-based method to determine optimal design of steel frame structures. The detailed procedure can be divided into the following two steps:

Step 1: Initialization. HS algorithm parameters such as HMS, HMCR, PAR, maximum number of searches and design variable are initialized. Harmonies (i.e., solution vectors) are then randomly generated from the possible variable bounds that are equal to the size of the HM. Here, the initial HM is generated based on a finite element analysis (ANSYS) subjected to the objective function and penalized objective function.

Step 2: Search. A new harmony is improvised from the initially generated *HM* or possible variable values using the *HMCR* and *PAR* parameters. These parameters are introduced to allow the solution to escape from local optima and to improve the global optimum prediction in the HS algorithm. The new harmony is analyzed using the finite element analysis (ANSYS), and its fitness is evaluated using the constraint functions. If satisfied, the weight of the structure is calculated using the objective function. If the new harmony is better than the previous worst harmony, the new harmony is included in the *HM* and the previous worst harmony is excluded from the *HM*. The *HM* is then sorted by the objective function value. The computations terminate when the maximum number of the search criterion is satisfied. If not, this step is repeated.

VI Benchmark Design Examples.

Two design problems have been examined in the present study to implement the developed optimum design algorithms. The design of steel frames with semi-rigid connections were compared those of rigid connections under similar design requirements. These semi-rigid and rigid connections frames were analyzed linearly and non-linearly including P-Δ effect of beam-column members. In addition, two catalogs was used, Full Catalog Section (FCS) that contain all beam-column members with W40 to W8. These sections are considered the most practical sections for steel beams and columns. Selected Catalog Section (SCS) that contains two separate population section lists was used to search economic solutions. The first one is column catalog with the height/width ratio less than 2; the second one is a beam section list with the height/width ratio greater than 2.

The design algorithm aims at obtaining the minimum steel weight of frames by selecting a standard set of steel W-sections from the AISC standard sections. AISC Strength, displacement, deflection and size constraint for all members and lateral torsional buckling were imposed on frames [**1**]. A comparative study was carried out between the HS optimization results and the results obtained for similar frames optimized using Genetic Algorithm (GA) techniques published by Kameshki and Saka [**22**].

A Design of three-storey, two-bay steel frame.

The geometry and loading of a three-storey, two-bay frame are shown in Figure 2. The Modulus of Elasticity and Yield stress of the steel sections are 29,000 ksi and 36 ksi, respectively. The top storey and inter-storey sway (H/300) is limited to 1.44 inch, 0.48 inch, respectively. The allowable deflection for service imposed load (L/360) is considered 0.66 inch. The out-of-plane effective length factor for each column (Ky) is taken 1.0. The out of plane unbraced length (L/6) for beams is specified to be 40 inch. Bolt diameter and end plate thickness are taken to be 1 inch, 0.685 inch, respectively.

The following tuning parameters are applied in HS algorithm; the harmony memory size *(HMS)* and the harmony memory consideration rate (*HMCR*) are selected as 15 and 0.90, respectively. The pitch adjustment rate (*PAR)* selected as *0.45* and bandwidth (*bw*) with a randomly selected neighboring index of -2 or +2, for example, if x_i^{new} is W14X68, the neighboring index of -2 or +2 forms a list of W14 x 90, W14 x 82, W14 x 74, W14 x 68, W14 x 61, W14 x 53, W14 x 48. The algorithm selects a random neighboring section from the four sections, namely; W14 x 82, W14 x 74 or W14 x 61, W14 x 53). The maximum iteration (Max_{iter}) is 2500 i^{th} [**36**].

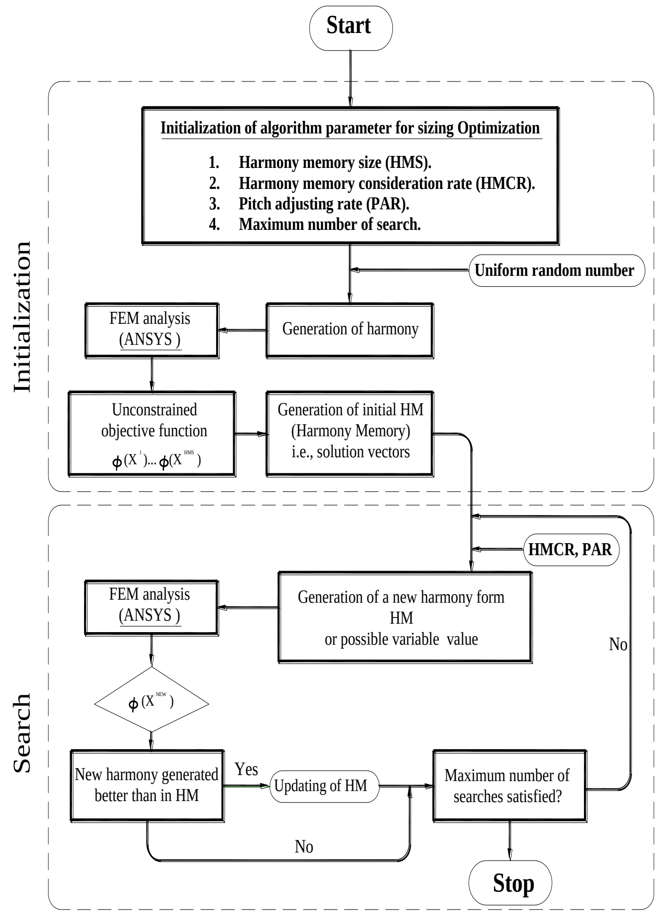


Figure 1 Harmony search algorithm optimization and design procedure.

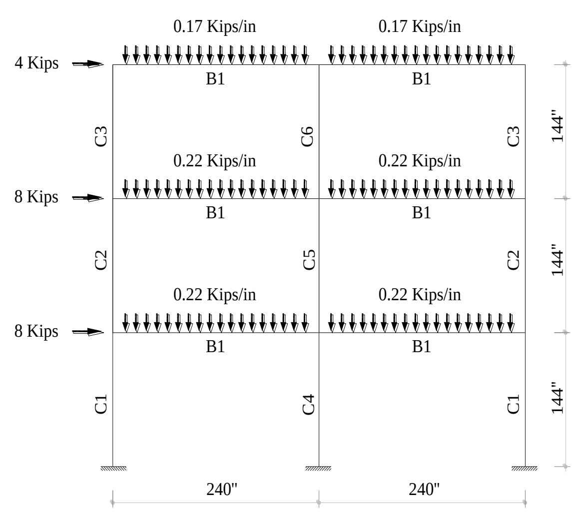


Figure 2 Three-storey, two-bay steel frame.

TABLE 2

Minimum steel frame weight of three-storey, two-bay steel frame based on HS.

Frame analysis No.	Rigid connection				Semi-Rigid connection					
	FCS				FCS				SCS	
	Linear		Non-linear		Linear		Non-linear		Non-linear	
	Weight Ib	**Iter. i^{th}**	**Weight Ib**	**Iter. i^{th}**	**Weight Ib**	**Iter. i^{th}**	**Weight Ib**	**Iter. i^{th}**	**Weight Ib**	**Iter. i^{th}**
1	6504	1746	6528	2235	6336	1579	6300	2366	6300	1547
2	6528	2199	6576	1392	6348	1802	6348	1255	6336	1513
3	6552	1578	6768	1621	6372	2488	6372	1135	6432*	1546
4	6624	1624	6792*	2235	6396	1615	6396*	1496	6432*	1249
5	6672	1125	6792*	1586	6456	1748	6396*	1490	6492	2005
6	6720	2303	6792*	1803	6816	2110	6504	1751	6504*	1558
7	6792	1542	6864	2130	6852	1657	6516	1622	6504*	1558
8	6804	1872	6888	2133	6876	1681	6648	1204	6744	1490
9	7116	2442	6924	1600	6912	1642	6696	2091	6756	1674
10	7248	2386	7272	1155	7020	2206	7128	1622	6852	1452
Min weight (Ib)	**6504**	-	**6528**	-	**6336**	-	**6300**	-	**6300**	-
MAPE (%)	**3.61**	-	**4.2**	-	**4.4**	-	**3.41**	-	**3.52**	-
Time (min)	**40**		**65**		**50**		**75**		**65**	

Note: 1- The * symbol have the same weights but different sections.

2- Mean absolute percentage error (MAPE) = $\frac{100\ \%}{n}\sum_{i=1}^{n}\frac{|\text{Actual value - Minimum value}|}{|\text{Actual value}|}$

Where, n = Frame analysis Number.

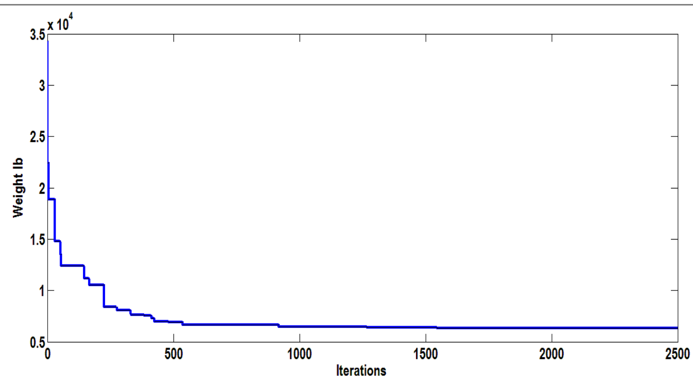


Figure 3. Optimum design history of three-storey, two-bay frame (SCS).

The results of ten independent runs of the HS steel design optimization algorithm are presented in Table 2. It is observed that the non-linear analysis including p-Δ effect of semi-rigid connection frames showed 3.50% less steel weight than those with rigid connections. Table 2 revealed that the optimum weight of semi-rigid connection with non-linear analysis in both Full Catalog Section (FCS) and Selected Catalog Section (SCS) has the same weight, but the convergence was obtained using only 65% of the expected iteration with FCS. This means that the SCS has an added flexibility in choosing beams and columns over those with FCS. In addition, the solution with linear analysis of semi-rigid connections yielded lighter frame weight than those with rigid connections 2.58% and the optimum weight converged at 1579^{th} iterations was obtained using only 63% of the expected maximum iteration

TABLE 3

Optimum design results, of three-storey, two-bay steel frame.

Three story, two bay frame		GAs (Kameshki , Saka, 2003)				HS (khalifa, 2011)				
Group	**Member type**	**Rigid connection**		**Semi-rigid connection**		**Rigid connection**		**Semi-rigid connection**		
		FCS		**FCS**		**FCS**		**FCS**		**SCS**
		Linear	**Non-linear**	**Linear**	**Non-linear**	**Linear**	**Non-linear**	**Linear**	**Non-linear**	**Non-linear**
1	Column	W24x55	W24X55	W21x50	W18X36	W21X44	W21X48	W12X30	W18X40	W12X35
2	Column	W21x44	W16X31	W18x35	W14X26	W14X30	W12X26	W12X26	W12X26	W12X26
3	Column	W12x26	W12X40	W18x35	W8X18	W10X22	W10X22	W8X24	W8X21	W8X24
4	Column	W30x108	W18X35	W27x84	W24X68	W14X38	W16X40	W14X48	W16X40	W14X43
5	Column	W24x55	W18X35	W24x55	W24X68	W14X30	W12X30	W12X30	W12X30	W12X30
6	Column	W18x35	W12X35	W18x46	W18X35	W10X22	W10X22	W12X30	W8X21	W10X22
7	Beam	W14x26	W16X26	W18x35	W16X26	W16X26	W16X26	W16X26	W14X26	W16X26
Total weight (Ib)		**8496**	**7404**	**9300**	**7092**	**6504**	**6528**	**6336**	**6300**	**6300**
% weight decrease		**25.42**	**14.91**	**31.87**	**11.16**	**2.58**	**3.50**	**0**	**0**	**0**
Top story sway		**0.48**	**0.64**	**0.39**	**0.61**	**0.78**	**0.63**	**1.13**	**0.93**	**0.92**

Note: Allowable top storey sway 1.44 inch

(Max_{iter}), 2500. Moreover, Table 2 reveals that in all cases of the HS mean absolute percentage error (MAPE) was obtained 3.40% - 4.40%, which reflected the accuracy of algorithm technique.

Figure 3 shows a typical convergence history for an HS design of the three-storey, two-bay steel frames with semi-rigid connections for SCS analysis. This Figure further illustrates that the optimization process decreased gradually to fine-tune. Because the values of pitch adjustments *PAR* and neighboring bandwidth (*bw)* decreased with time to prevent overshoot, oscillations and forcing the algorithm to focus more on intensification in the final iterations. Furthermore, the Figure revealed that the convergence curve was high up to the iteration number 500^{th}, and the optimum value was obtained at the iteration number 1547^{th} and kept unchanged until the maximum iteration is reached (Max_{iter}), 2500.

B Comparison of HS with genetic algorithm for three-storey, two-bay steel frame.

The optimum steel section designations obtained by the Harmony Search (HS) method is given in Table 3. The optimum weight of frames with semi-rigid connections is generally less than that of frames with rigid connections. In addition, the optimum weight of semi-rigid connection with non-linear analysis in both FCS and SCS has the same weight, but the sections are completely different. When comparing the results of HS with the corresponding frames optimized using genetic algorithm (GA) technique, the HS indicated 11.16% lighter weights than those optimized using GAs. Furthermore, Table 3 revealed that in all cases HS yielded lighter frames between 11.16% - 31.87% compared with those obtained by GAs.

The results also showed that the lateral displacement at the top storey was 0.92 inch in case of non-linear semi-rigid frame with SCS analysis, which is higher than those obtained by GAs, but within the allowable limit of AISC-LRFD (1.44 inch). This can be attributed to the fact that lighter sections sways more than heavier members.

C Ten-storey, one-bay steel frame.

The geometry and loading of a ten-storey, one-bay frame are shown in Figure 4. The Modulus of Elasticity and Yield stress of the steel sections are 29,000 ksi and 36 ksi, respectively. The top storey and inter-storey sway (H/300) is limited to 4.92 inch, 0.48 inch, respectively. The allowable deflection for service imposed load (L/360) is considered 1.00 inch. The out-of-plane effective length factor for each column (Ky) is taken 1.0. The out of plane unbraced length (L/6) for beams is specified to be 60 inch. Bolt diameter and end plate thickness are taken to be 1.125 inch, 1.00 inch, respectively.

The following tuning parameters are applied in HS algorithm; the harmony memory size *(HMS)* and the harmony memory consideration rate (*HMCR*) are selected as 20 and 0.90, respectively. The pitch adjustment rate (*PAR)* and neighboring bandwidth (*bw*) are selected as 0.45 and ± 2, respectively. The maximum iteration (Max_{iter}) is 5000 i^{th}. [**36**].

The results of ten independent runs of the HS steel design optimization algorithm are presented in Table 4. It is observed that the linear analysis of semi-rigid connection frames showed 2.03% less steel weight than those with rigid connections. Table 4 revealed that the optimum weight of semi-rigid connection with linear analysis using SCS showed 1.87% less steel weight than those with FCS. This means that the SCS has flexibility in choosing beams and columns than those with FCS. Furthermore, the solution with non-linear analysis of semi-rigid connections resulted in a heavier frame weight than those with rigid connections 1.40% due to the magnitude of loading and frame configuration. Over the above, Table 4 revealed that in all cases of the HS mean absolute percentage error (MAPE) was obtained 1.78% - 6.21%, which reflected the accuracy of algorithm technique.

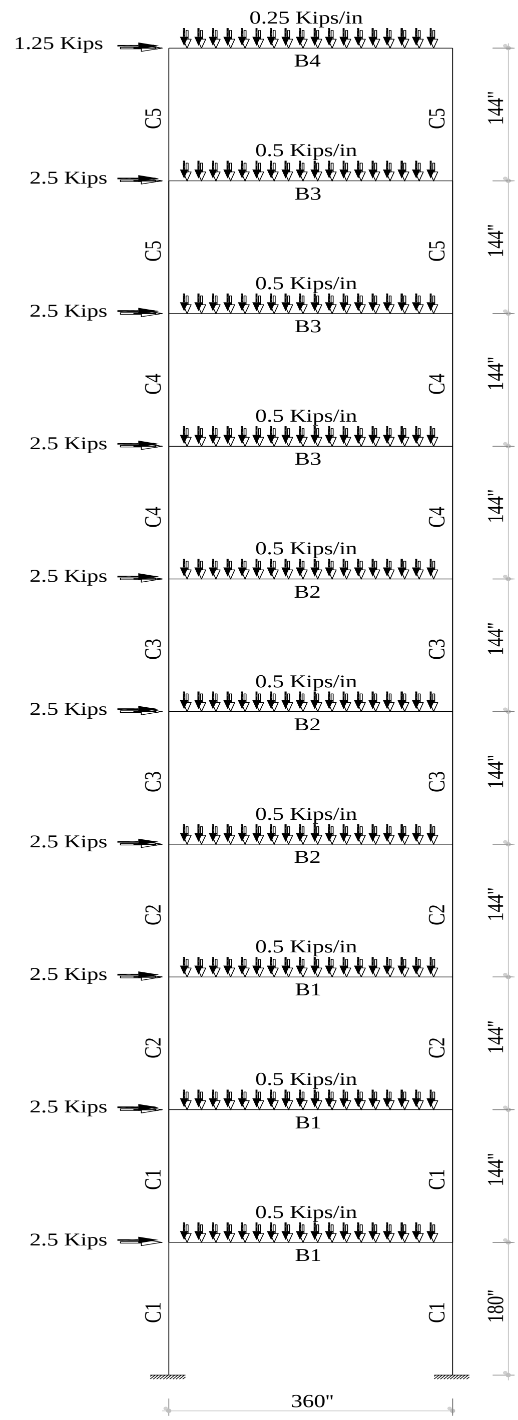


Figure 4 Ten-storey, one-bay steel frame.

Figure 5 shows a typical convergence history for an HS design of the ten-storey, one-bay steel frames with semi-rigid connections for SCS analysis. As shown in this Figure, the optimization process decreased gradually to fine-tune. Because the values of pitch adjustments *PAR* and neighboring bandwidth (*bw)* decreased with time to prevent overshoot, oscillations and forcing the algorithm to focus more on intensification in the final iterations. Also, the Figure revealed that the convergence curve was high up to the iteration number 1000^{th}, and the optimum value was obtained at the iteration number 4050^{th} yet remained unchanged until the maximum iteration (Max_{iter}), 5000.

TABLE 4

Minimum steel frame weight of ten-storey, one-bay steel frame based on HS.

Frame analysis No.	Rigid connection				Semi-Rigid connection					
	FCS				FCS		SCS		FCS	
	Linear		Non-linear		Linear		Linear		Non-linear	
	Weight Ib	Iter. i^{th}	Weight Ib	Iter. i^{th}	Weight Ib	Iter. i^{th}	Weight Ib	Iter. i^{th}	Weight Ib	Iter. i^{th}
1	48828	4122	48420	4421	48744	3492	47832	4050	49110	4864
2	48972	4919	48654	3727	49068	4994	48216	3916	49146	4274
3	48984*	4244	48750	3815	49134	3627	49956	3186	49158	4175
4	48984*	4244	48972	4255	49248	2954	50082	4250	49302	3492
5	49086	3105	49242	4009	49734	4690	50484	2933	49464	4772
6	49230	4908	49596	4671	50316*	3323	51996*	2670	50748	3590
7	49242	2468	49668	4226	50316*	3323	51996*	2670	50508	4894
8	50106	3674	49908	3479	51792	2948	52206	4190	51132	3489
9	51600	2071	50166	4872	51810	4908	54000	3074	51168	4442
10	52368	2982	52254	4326	52428	4395	54042	4155	51912	3814
Min weight (Ib)	**48828**	**-**	**48420**	**-**	**48744**	**-**	**47832**	**-**	**49110**	**-**
MAPE (%)	**1.78**	**-**	**2.26**	**-**	**2.95**	**-**	**6.21**	**-**	**2.06**	**-**
Time (min)	**75**		**160**		**90**		**100**		**175**	

Note: The * symbol have the same weights but different sections.

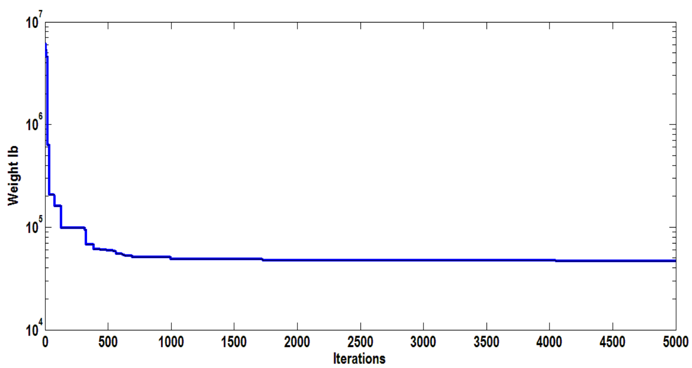


Figure 5. Optimum design history of ten-storey, one-bay frame (SCS).

D Comparison of HS with genetic algorithm for ten-storey, one-bay steel frame.

The optimum steel section designations obtained by the Harmony Search (HS) method is given in Table 5. The optimum

TABLE 5

Optimum design results, of ten-storey, one-bay steel frame.

Ten-story, one-bay frame		GAs (Kameshki , Saka, 2003)				HS (khalifa, 2011)				
Group	Member type	Rigid connection		Semi-rigid connection		Rigid connection		Semi-rigid connection		
		FCS		FCS		FCS		FCS	SCS	FCS
		Linear	Non-linear	Linear	Non-linear	Linear	Non-linear	Linear	Linear	Non-linear
1	Column	W36x135	W36x182	W36x160	W36x182	W36X150	W36X150	W24X162	W27X146	W33X152
2	Column	W33x141	W36x135	W36x135	W36x135	W30X132	W33X130	W24X131	W21X122	W30X132
3	Column	W30x108	W30x108	W36x135	W33x118	W27X114	W33X118	W21X101	W21X101	W30X108
4	Column	W27x102	W24x68	W33x118	W27x102	W24X84	W27X84	W14X82	W18X76	W30X90
5	Column	W14x90	W21x111	W30x108	W14x99	W18X76	W24X68	W14X68	W14X82	W27X84
6	Beam	W24x68	W24x68	W24x68	W33x118	W24X76	W24X76	W24X68	W24X68	W24X68
7	Beam	W24x68	W24x68	W24x68	W24x76	W24X76	W24X76	W24X68	W24X68	W24X68
8	Beam	W27x84	W24x68	W24x68	W21x93	W24X68	W24X68	W27X84	W27X84	W24X76
9	Beam	W30x108	W21x44	W18x35	W18x50	W21X48	W21X44	W21X62	W21X62	W18X65
Total weight (Ib)		**51498**	**49764**	**51858**	**58950**	**48828**	**48420**	**48744**	**47832**	**49110**
% weight decrease		**7.11**	**1.31**	**7.76**	**16.70**	**2.03**	**-**	**1.87**	**0**	**0**
Top story sway		**0.93**	**1.35**	**1.21**	**1.43**	**0.91**	**1.28**	**1.45**	**1.43**	**1.96**

Note: Allowable top storey sway 4.92 inch

weight of frames with semi-rigid connections is generally less than that of frames with rigid connections. When comparing the results of HS with the corresponding frames optimized using genetic algorithm (GA) technique, the HS indicated 7.76% lighter weights than those optimized using GAs. Furthermore, Table 5 revealed that in all cases HS yielded lighter frames between 1.31% -16.7% compared with those obtained by GAs.

The results also showed that the lateral displacement at the top storey was 1.43 inch in case of linear semi-rigid frame with SCS analysis, which is higher than those obtained by GAs, but within the allowable limit of AISC-LRFD (4.92 inch). This can be attributed to the fact that lighter sections will sway more than heavier members.

VII CONCLUSION.

The optimum design algorithm is developed for semi-rigid steel frames based on the harmony search method which is a new stochastic random search approach that simulates the musical process of searching for a perfect state of harmony. The benchmark design examples presented in this study revealed that the designs with semi-rigid connection resulted in lighter frames than the ones with rigid connections. In addition, it is observed that nonlinear semi-rigid frames are lighter in some cases and heavier in some others, compared to linear semi-rigid frames, depending on the magnitude of loading and frame configuration.

The results obtained showed that the harmony search method is an efficient and robust technique, because it reached lighter frame sections than GAs 1.31%- 31.87%. Moreover, The Optimization using Selected Catalog Section (SCS) resulted in lighter frame sections than using Full catalog section (FCS) about 1.87%. It is further noticed that the mean absolute percentage error (MAPE) was obtained 1.78% - 6.21%. Furthermore, the maximum sway obtained at the optimum design increased smoothly in case of semi-rigid frame in compression with a rigid frame, this can be attributed to the fact that lighter sections sway more than heavier members yet remain within the allowable limit by AISC-LRFD.

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Dr. Mohammed Arafa is associate professor in structural engineering at faculty of engineering- Islamic University of Gaza. Dr. Arafa has wide experience in structural analysis, finite element method, computational mechanics and optimization of structures.

Ashraf Khalifa. has M.Sc. dgree in civil engineering, The Islamic University of Gaza, Palestine.

Dr. Mamoun Alqedra is associate professor in civil engineering at faculty of engineering- Islamic University of Gaza. Dr. Alqedra has wide experience in design and analysis of steel structure, geotechnical engineering, computational mechanics, and optimization of structures.